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ACCURACY COMPARISON OF TOTAL STATION (TS) AND GLOBAL POSITIONING SYSTEM (GPS) IN DETERMINING HEIGHT WITHIN AN URBAN ENVIRONMENT

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ABSTRACT

Positioning accuracy of earth surface points is usually needed for various surveying and engineering projects. All the traditional methods of spatial measurement such as theodolites and differential levels give highly acceptable accurate spatial data. They are however, time consuming and consequently expensive. The use of modern technology such as GPS and Total Station is becoming favorite in the field of spatial measurement. For this reason, such techniques of point poisoning are becoming widely used. Considerable research results showed that GPS horizontal positioning accuracy compete well with Total Station. The objective of this paper, however, is to evaluate and compare accuracy of Total Station (TS) and Global Positioning System (GPS) in height determination within an urban environment. Network of height control points for accuracy comparison of these two techniques was established using a digital level.Test results show that GPS is 5 times less accurate than TS in elevation determination. Height accuracy obtained using TS was 0.004m, while height accuracy obtained using GPS in Fast Static mode was 0.020m in an urban environment for a nine points traverse of total distance around 400m.

KEYWORDS: Accuracy; height; GPS; Total Station; Control Network; Standard Deviation.

1. INTRODUCTION

One of the main objectives of surveying is the determination of the terrestrial or three-dimensional position of points on the earth surface. These points are usually used to establish control points for topographic or detailed maps and boundaries for cadastral survey. Topographic mapping using digital photogrammetry or any other approach requires ground control established in three dimensions.

Topographic maps are important for various applications such as planning and construction of engineering projects related to ground surface, geological exploration, hydrological and meteorological services and submerged topography or bathymetry (Akpinar and Aykut, 2017).

Contour maps indicating the shape and elevation of the land over the entire parcel is usually required for many applications, some of which are given below

- Spot heights covering the entire survey area showing high points, low points, grade changes, and representation of the general character of the terrain.
- Longitudinal and cross sections for earthwork calculations
- Main floor elevations of buildings.
- Elevations of lakes, rivers, streams or drainage courses.
- Elevation of bench marks used as control for the survey.

One of the output of three dimensional spatial measurements is the digital elevation model (DEM). DEMs support all construction activities and infrastructure engineering in urban and rural environments, as well as mapping and monitoring the natural environment.

The vertical precision is also important for many applications (Lin, 2004; Kutalmis, et al, 2017; Alizadeh-Khameneh, et al, 2018; Weaver, et al, 2018).

The fundamental basics of land surveying have been in use for long time. One of the most important elements that affect the accuracy of land surveys are the tools that the land surveyors use. The tools and apparatus used in surveying, however, have passed drastic development that has really improved the accuracy of land surveys.

One of the most popular and preferred tools used to ensure the accuracy of land surveys is the total station (TS). The TS is an electronic theodolite which includes an electronic distance measurement device (EDM) and a processing unit. GPS systems are also preferred and used to give high accuracy of land surveys. GPS systems however do not work well in areas crowded with buildings and with dense tree cover or construction areas (Pirti, et al, 2009; Schloderer, et al, 2017).

Considerable research results showed that GPS horizontal positioning accuracy compete well with Total Station. On the other hand the height accuracy obtained using GPS is debatable and not always satisfactory (Ahmed, 2000; Choi, et al, 2007; Kizil and Tisor, 2011; Diwakar et al, 2014; Sama and Stombaugh, 2014). In the following section a summary of some tests carried out to investigate height accuracy for TS and GPS will be outlined.

2. LITERATURE REVIEW AND PAPER OBJECTIVE

During the last three decades, a lot of research have been made in analyzing accuracy of both TS and GPS. Some of the results of such work will be outlined in this section in order to give brief idea about the overall concept of accuracy of the two techniques: TS and GPS:

- In order to check the compatibility of the GPS RTK method with that of total station method, *Ahmed, (2000)* tested GPS RTK and total station measurements on an existing network. According to the result, the difference between the coordinates of total station and GPS RTK was 2 cm for the horizontal and 3 cm for the vertical coordinates.
- According to *Featherstone and Stewart, 2001* a 60-point test network, established on Curtin University of Technology's Bentley Campus in Perth, Western Australia, has been used to evaluate the accuracy of three different models of RTK GPS equipment as used by three different contractors. Therefore, these evaluations assess the combined performance of the RTK GPS equipment as used by each contractor, since they are inextricably linked. No evidence of any vertical bias in the test network was detected, and the estimated error in the control WGS84 ellipsoidal heights of 15 mm (95% confidence) appears to be realistic.
- *Ceylan, et al 2005* carried out tests to compare height accuracy obtained by different techniques: geometric levelling using Automatic level with a wooden rod, a Digital level with a bar coded rod, Trigonometric levelling using a theodolite, a Total Station and a GPS. The root mean square errors obtained from using these techniques were respectively: ±3.7mm, ±2.0mm, ±16.4mm, ±14.7mm and ±18.8mm. The last two results show that Total station height accuracy is better than GPS height accuracy by 25%.
- *Saghravani, et al, 2009* investigated accuracy comparison of RTK-GPS and automatic level for height determination in the vicinity of University of Putra Malaysia campus.

Their results indicate that accuracy of 0-10 cm in the measurement of elevation by RTK-GPS can be obtained.

- *Chekole, 2014* carried out a test to compare accuracy and time expenditure of total station, RTK GPS and terrestrial laser scanner (TLS) using a reference network composed of 14 control points established by using a Leica 1201 TS. Tet results revealed that TS giving 1 mm for both horizontal and vertical coordinates, while RTK GPS gave 9 mm in horizontal and 15mm accuracy in vertical coordinates.

The objective of this paper is to present test height accuracy of TS and GPS that had been carried out in an open area within an urban environment, KSU campus, where a net of nine control points were established using a differential digital level.

Methodology of the test including test site and instruments used will first be outlined. Test operations, results, analysis and conclusions will then follow.

3. METHODOLOGY

In order to evaluate the height accuracy of the instruments under test (a Total Station and a GPS), a network of control points which can serve as a reference for the comparison was established. The reference network established is composed of nine control points selected in a limited open area within King Saud University campus, north of Riyadh, capital of Kingdom of Saudi Arabia. Sokkia Digital Level SDL30 was used to establish levels of the control network.

3.1. Test Site

The Kingdom of Saudi Arabia (KSA) occupies about 80% of the Arabian Peninsula (the world's largest peninsula), lying between latitudes 16° and 33° N, and longitudes 34° and 56° E. The estimated area of KSA is about 2,149,690 Km^2. Riyadh, capital of kingdom of Saudi Arabia, lies in the center of Arabian Peninsula on latitude 34° - 38' north and longitude 46^o – 43' east approximately 600 meters above Mean Sea Level in Eastern Najd, a region largely dominated by a rocky plateau landscape, in the center of the Arabian Peninsula (Figure 1).

Fig. 1: Saudi Arabia Map and Riyadh Location.

Geographic information of Riyadh city are given in table 1 below:

Table 1: Riyadh Geographic information.

Latitude	24° 46' 27.35" N
Longitude	46° 44' 18.91" E
Elevation (m)	638 m
Category	Cities
Country	Saudi Arabia

The site test is an area within King Saud University (KSU) which falls in the north part of the city of Riyadh, the capital of KSA. (Figures. 2a and 2b).

Fig. 2a: King Saud University Campus. **Fig. 2b: Test Area bounded by red line.**

Figure 3 shows the selected nine points traverse connected with traverse lines. The site is a clear open space with few neighboring buildings.

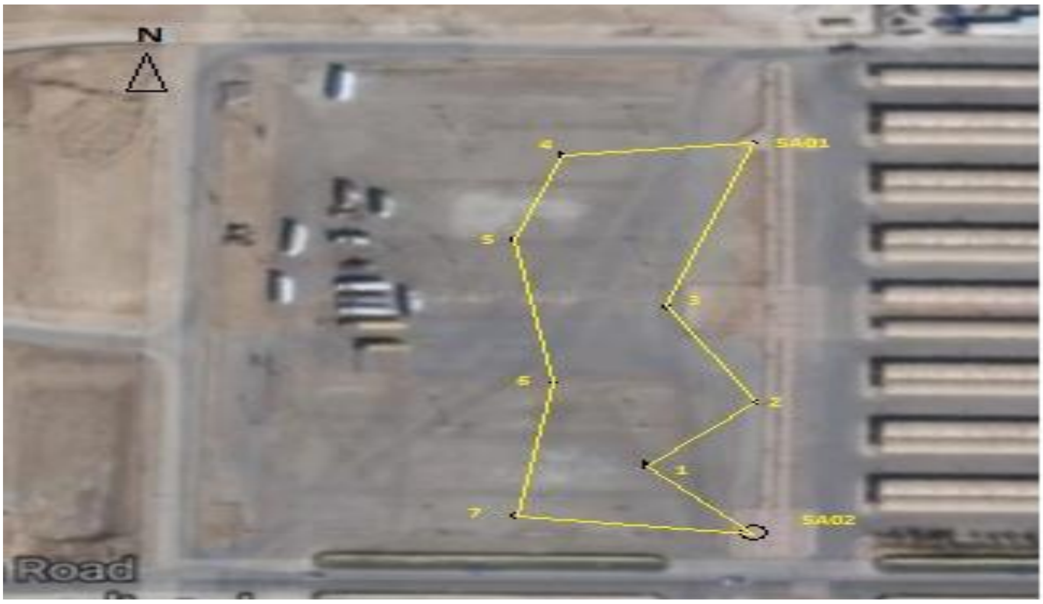

Fig. 3: Test Site and Control Traverse.

3.2 Test instruments and Techniques

A digital level to determine levels of the nine-points traverse test, while a TS and a GPS were tested to evaluate the accuracy of each. These instruments will be introduced in the coming sections.

Differential Digital Level

Leveling is a process of finding the elevation of a given point with respect to the given or assumed datum. There are three leveling techniques: differential, trigonometric, and barometric. Differential leveling is the most accurate of the three methods. With the instrument locked in position, readings are made on two calibrated staffs held in an upright position ahead of and behind the instrument. The difference between readings is the difference in elevation between the points. Trigonometric leveling involves measuring a vertical angle from a known distance with a theodolite and computing the elevation of the point. With this method, vertical measurements can be made at the same time horizontal angles are measured for triangulation. It is, therefore, a somewhat more economical method but less accurate than differential leveling. It is often the only practical method of establishing accurate elevation control in mountainous areas. In barometric leveling, differences in height are determined by measuring the differences in atmospheric pressure at various elevations. Air pressure is measured by mercurial or aneroid barometer, or a boiling point thermometer. Although the accuracy of this method is not as great as either of the other two, it gives relative heights very rapidly at points which are fairly far apart.

There are different models of differential levels. From the point of view of technical design they can be classified as Optical and Electronic (digital) levels.

The differential level used in this project to provide the height of points of control network is the digital Sokkia SDL30, with Fiberglass RAB-Code Staff, of accuracy claimed to be ±1mm for 1 km double run levelling (Figure 4).

Fig. 4: Sokkia SDL 30 Digital Level.

Total station

A TS is a modern surveying instrument that integrates an electronic theodolite with an electronic distance meter (EDM), in addition to a processing unit.

In this project, Total Station Leica TC 407 (Figure 5) with distance accuracy of 2mm ±2ppm and angular accuracy of 7” available in the civil engineering department surveying laboratory, KSU has been selected to be used in height accuracy test.

Fig. 5: Leica TC407 Total Station.

GPS

The Global Positioning System (GPS) is a space based radio navigation system that provides geolocation and time information to a GPS receiver anywhere on or near the Earth Surface where there is an unobstructed line of sight to four or more GPS satellites. The GPS used in this test is Leica 500 (Figure 6). The main components of System 500 are the AT502 dual frequency antenna and SR530 Receiver. Ancillary components are the Terminal, Batteries, PC Cards and cables.

Fig. 6: Leica 500 GPS

Techniques of GPS measurement that are utilized by surveyors

Static GPS is used for determining accurate coordinates for survey points by simultaneously recording GPS observations over a known and unknown survey point for at least 20 minutes. The data is then processed in the office to provide coordinates with an accuracy of better than 5mm depending on the duration of the observations and satellite availability at the time of the measurements. This procedure allows various systematic errors to be resolved when high-accuracy positioning is required. Static GPS surveying is a relative positioning technique which employs two (or more) stationary receivers simultaneously tracking the same satellites. One receiver, the base receiver, is set up over a point with precisely known coordinates such as a survey monument. The other receiver is set up over a point whose coordinates are unknown. This method of surveying is based on collecting simultaneous measurements at both receivers for a certain period of time, which, after processing, yield the coordinates of the unknown point. This type of survey is primarily used to create control where no control exists to very high accuracies (Anquela, et al, 2013). To derive ellipsoid heights on passive marks with centimeter-level accuracy, many current specifications require the collection and adjustment of long-duration, static, post-processed global navigation satellite system (GNSS) sessions (Weaver, et al, 2018).

Fast-static GPS surveys are similar to static GPS surveys, but with shorter observation periods (approximately 5 to 10 minutes). Similar to Static mode, it is a carrier-phase-based relative positioning technique employing two or more receivers simultaneously tracking the same satellites. However, with rapid static surveying, only the base receiver remains stationary over the known point during the entire observation session while the rover receiver remains stationary over the unknown point for a short period, and then moves to another point whose coordinates are to be determined (Bakula, 2012). **This method is suitable when the survey involves** a number of unknown points located within the survey area. **After collecting and downloading the field** data from both receivers, the PC software is used for data processing giving the coordinates of the unknown points.

Advantages of Fast Static mode include

- Observation time is dependent on (a) length of the baseline (b) number of visible satellites
- No continuous locking is required with moving rover from one station to the next since each baseline is processed independent of each other. In fact, receivers can be turned off to preserve batteries (though not recommended).
- It may be noted that the initial phase ambiguities can be resolved within a minute for a dual frequency receiver (3-5 minutes for single frequency receivers).
- The method does not require re-observation of remote stations like pseudo-kinematic or reoccupation method.
- Accuracies : Similar to static: ± (5-10 mm + 1 ppm); 1:100,000 to 1:1,000,000
- Applications of this mode includes: fixing control surveys, detail surveys as well as replacing traversing and ground triangulation.
- Advantages: easy, quick, efficient and ideal for short range survey.

Real Time Kinematic (RTK) Observations: This is where one receiver remains in one position over a known point – the Base Station – and another receiver moves between positions – the Rover Station. The position of the Rover can be computed and stored within a few seconds, using a radio link to provide a coordinate correction.

Table 2: Comparison between Static and Rapid Static GPS Survey.

GPS Mode	Observation Time	Applications	Accuracy
Static	Long	Control network	Higher accuracy
Rapid Static	Shorter	Many Survey Applications: engineering, cadastral	Lower Accuracy

3.3 Establishment of Height Control Net

A geodetic control network is a network, usually of triangles whose vertices positions are precisely determined using terrestrial survey methods.

It consists of stable, identifiable points with published datum values derived from observations that tie the points together.

The digital level Leica SLD 30 was used to determine levels of the traverse net points already shown in Figure 3 with points SAO1 and SAO2 with levels 658.827m and 657.438m, respectively, (Table 3) used as bench marks.

Table 3: Coordinates of Control Points SA01 and SA02.

Point	East (m)	North (m)	Elevation (m)
SAO1	663230.700	2735312.290	658.827
SAO2	663230.593	2735177.595	657.438

The closure error of height observation was 0.001m. Elevations of traverse points were adjusted to close with zero error given in Table 4. These were then used to compare accuracy of elevations of traverse points obtained by using TS and GPS.

Table 4: Traverse control points (Elevations determined using Digital Level).

Point	Elevation (m)
SA02	657.438
1	657.575
2	658.122
3	658.699
SA01	658.827
4	658.500
5	658.622
6	658.016
7	656.991

4. TESTS AND RESULTS

4.1 Total station Test and results

The Total Station was centered on the control point SA02. Its coordinates were fed in and the instrument telescope was directed towards the second point SA01, hence recording azimuth of the reference line. Test points whose levels were already determined using the precise digital level were observed in turn and their coordinates were recorded as in Table 5. Total station test results including difference from the precise level results, minimum, maximum, standard deviations and rmse are given in Table 5.

Table 5: Test points (Elevations determined using TS compared to those determined from Digital Level).

Point	Elevation (m) Digital Level	Elevation (m) TS	dh (m) TS
SA02	657.438	657.438	0.000
1	657.575	657.574	-0.001
2	658.122	658.125	0.003
3	658.699	658.703	0.004
SA01	658.827	658.833	0.006
4	658.500	658.506	0.006
5	658.622	658.625	0.003
6	658.016	658.022	0.006
7	656.991	656.992	0.001
Min			0.001
Max			0.006
Mean			0.004
Standard Deviation, σ			0.004
rmse			0.004

4.2 GPS TEST AND RESULTS

Rapid Static mode was used in this test. Steps of measurement are summarized as follows:

- Before data collection planning was carried out to observe theoretical satellite availability. Most GPS software has the ability to provide a theoretical estimate of the satellite availability at a given location and time. Data collection is hence planned to be done at times when there is optimum satellite availability and when the satellites are at appropriate configuration to produce an acceptable (lower) PDOP value. Using Satellite Availability program the suitable dates and intervals for Observations were selected from the charts shown below as given for both stations SA01 and SA02: Figures 7 and 8 show observation date plan, while Table 6 shows observation specifications for SA01 and SA02, respectively.

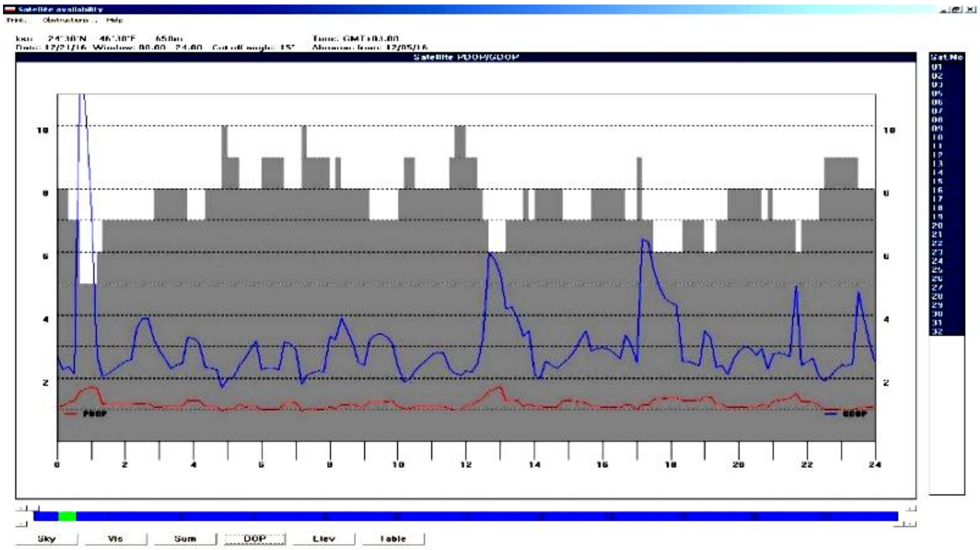

Fig. 7: SA01 Observation date plan.

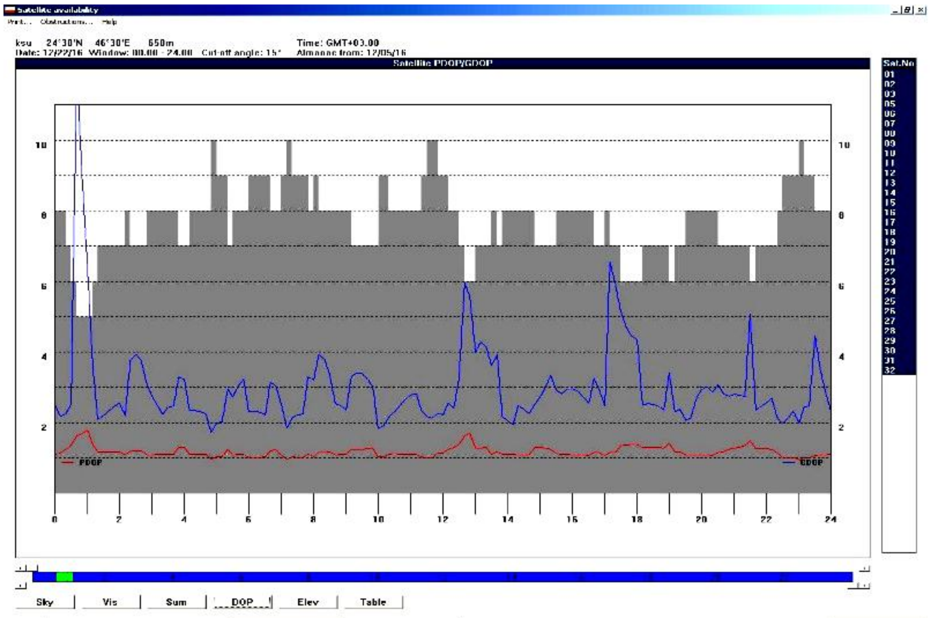

Fig. 8: SA02 Observations date plan.

Table 6: GPS Observations specifications.

Point	GDOP	PDOP	Period	Interval	Number of visible satellites
SA01	Max 3.3	Max 1.8	3 Hours	8.35 – 11.48	10
SA02	Max 3.9	Max 1.5	3 Hours	7.43 – 10.45	10

- Set the reference receiver at the reference point (control point: SAO2)

- Set the rover at each point of the nine traverse points for 5 minutes.

- Process the measured data by SKI Program.

Results obtained from GPS test are given in Table 7.

Table 7: GPS Test Results.

Point	Elevation (m) Digital Level	Elevation (m) GPS	dh (m) GPS
SA02	657.438	657.438	0.000
1	657.575	657.606	0.031
2	658.122	658.140	0.018
3	658.699	658.704	0.005
SA01	658.827	658.852	0.025
4	658.500	658.519	0.019
5	658.622	658.624	0.002
6	658.016	658.036	0.020
7	656.991	656.986	-0.005
Min			0.002
Max			0.031
Mean			0.015
Standard Deviation, σ			0.020
rmse			0.019

It is noticed that the errors in both results were in one direction except for point 1 in TS results and point 7 in GPS results. Subtracting a constant value equal to the standard deviation from each computed height will improve the results as shown in the table. The standard deviation of height from TS will be 0.0026m and from GPS will be 0.0137m.

5. ANALYSIS OF RESULTS

Results of both instruments, TS and GPS as compared to the Digital Level are summarized in Table 8 below.

Table 8: TS and GPS Test Results.

Point	Dh (m) (TS)	Adj=-0.004m	dh (m) (GPS)	Adj=-0.020m
SA02	0.000		0.000	
1	-0.001	-0.005	0.031	0.011
2	0.003	-0.001	0.018	-0.002
3	0.004	0.000	0.005	-0.015
SA01	0.006	0.002	0.025	0.005
4	0.006	0.002	0.019	-0.001
5	0.003	-0.001	0.002	-0.018
6	0.006	0.002	0.020	0.000
7	0.001	-0.003	-0.005	-0.025
Min	0.001	0.001	0.002	0.001
Max	0.006	0.005	0.031	0.025
Mean	0.004	0.002	0.015	0.008
σ	**0.004**	**0.003**	**0.020**	**0.014**
rmse	**0.004**	**0.003**	**0.019**	**0.013**

From the above table of results the maximum and minimum height closure errors when using TS are 6mm and 1mm respectively, while the rmse of heights is ±3mm. For the GPS the maximum and minimum height errors are 31mm and 2mm, respectively, and the rmse is ±13mm. This means that the height accuracy of the GPS is almost 1/4th of the TS height accuracy. This result confirms what is usually stated by other researchers when comparing height accuracy of TS and GPS.

California Department of Transportation (CARLTRANS), 2015 adopts the minimum closure standards for vertical controls set by Federal Geographic Data Committee (FGDC) which is given in Table 9 below.

Table 9: Minimum Elevation Closure Standards for Vertical Control Surveys.

Classification Order	Elevation Closure Standard
First Order, Class I	$3\sqrt{k}$ (mm)
First Order, Class II	$4\sqrt{k}$ (mm)
Second Order Class I	$6\sqrt{k}$ (mm)
Second Order, Class II:	$8\sqrt{k}$ (mm)
Third Order	$12\sqrt{k}$ (mm)
Construction Layout	$24\sqrt{k}$ (mm)

Comparing the height accuracy obtained using GPS Fast Static Mode with the elevation closure standard set by FGDC, it can be deduced that GPS can easily be used for height determination for construction works, since it satisfies a height closure of $20\sqrt{k}$

USGS topographic maps adhere to "National Map Accuracy Standards". The vertical accuracy standard requires that the elevation of 90 percent of all points tested must be correct within half of the contour interval. On a map with a contour interval of 1.0 m, the map must correctly show 90 percent of all points tested within 50 cm of the actual elevation. Hence GPS Fast Static can satisfactorily be used to collect data for 1.0m contour interval maps.

6. CONCLUSIONS AND RECOMMENDATIONS

The purpose of this research is to compare accuracy in elevation between Total Station and GPS where the references are a set of traverse points whose elevations were determined using a digital level.

A network of nine control points was constructed using the digital level, with 2 control points already existing. Test of GPS was based on Fast-Static mode.

The obtained results from this test confirmed previous results about height accuracy of the Total Station and GPS in fast static mode. One can differentiate which instrument should be used for which specific application depending on the presented results and the project requirements.

GPS Fast Static can fairly be used for height determination for construction works where the minimum height standard error is to be less than $24\sqrt{k}$.

For survey applications which require high accuracy to serve as height reference, such as height control point establishments, it is recommend to use TS instead of GPS.

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