PHILIPPINE ENGINEERING JOURNAL
PEJ 2015; Vol. 36, No. 2: 1-20

Applicability and Implications of the Use of Real Time Kinematic GNSS for Property Surveys in the Philippines

Louie P. Balicanta
Assistant Professor, Department of Geodetic Engineering
University of the Philippines Diliman, Quezon City, Philippines 1101

***Abstract*— Property Survey is the use of surveying techniques and procedures to obtain the location, size and shape of a land parcel that is owned or claimed by individual or group entities. It is both technical and legal in nature. Real Time Kinematic (RTK) Global Navigation Satellite System (GNSS) method is the use of dual-frequency GNSS to obtain the position of an object/ entity in real time. Presently, this is widely used in the conduct of surveying tasks such as topographic surveys and stake-out activities in construction surveying. Its use in property survey is not a straightforward application and must consider different situations, problems and limitations. Factors to consider includes coordinate systems used in the country, concepts of common point, azimuths and distances, and limitations such as multipath and obstructions. Methodologies and tests include azimuth determination, survey under tree canopy and use for old and new surveys. The results of the experiments show that RTK-GNSS is applicable for parcel corner position determination with consideration to limitations such as obstructions, level of accuracy, systematic error from projection, different coordinate systems used in the country and poor identification of common points.**

Keywords*— *RTK-GNSS, Rapid Static GNSS, Property Survey, PRS92, Technical Descriptions

1. INTRODUCTION

1.1 Rationale

Property survey or land survey is traditionally defined as the measurement and lay-out of directions and lengths of lines to form the boundaries of a real property (Anderson & Mikhail, 1998). The main purposes of property survey are (a) to provide technical description of a land property as an input to ownership document such as land title, (b) to re-establish boundary monuments and (c) to consolidate land parcels, and/ or subdivide a parcel, producing new boundaries.

Traditional land surveying techniques make use of optical instruments such as transits and theodolites combined with an engineer's tape measure and electronic total stations wherein distances and directions are measured using methods such as traverse and side-shots. The final outputs include positions of the boundary corners and areas of the land parcels usually in units of square meters.

GNSS provides an alternative way of conducting surveys as it provides positions of fixed and moving objects through the use of positioning satellites. Static differential technique is considered the

Correspondence to: Louie P. Balicanta, Department of Geodetic Engineering, University of the Philippines Diliman, Quezon City, Philippines 1101 | Email: *louie_balicanta@yahoo.com*

most accurate technique and is being used in the Philippines as the main process in the establishment of project controls for cadastral survey projects (DENR, 2010). However, the current practice to obtain parcel boundaries mainly uses optical instruments such as electronic total stations. RTK-GNSS is considered as alternative method.

Specific provisions on the use of RTK-GNSS for acquisition of technical descriptions for a land parcel were not included in the DENR Administrative order 2007-29: Manual for Land Surveying in the Philippines nor in the DENR Memorandum Circular 2010-13: Adoption of the Manual on Land Survey Procedures. However recently, DENR Land Management Bureau (LMB) issued Memorandum Circular No. 2015-001, "*Guidelines on the Use of Real Time Kinematic (RTK) Global Navigation Satellite System (GNSS) in the Conduct of All Kinds of Lot Surveys with Tertiary Accuracy"*.

The research is aimed at examining related literature and guidelines, considerations, applicability and implications of the use of RTK-GNSS for property survey in the Philippine setting to supplement and improve existing policies and guidelines. This paper reviews the use of RTK-GNSS, the concepts and practices that affect or will be affected by the use of RTK-GNSS, and proposes a methodology for the utilization of the technology for property surveys.

1.2 Related Concepts and Studies

1.2.1 Coordinate System

DAO 2007-29 mentioned three (3) coordinates systems used in the context of land surveying in the Philippines: namely, local plane coordinate system (LPCS), Philippine Plane Coordinate System Transverse Mercator/ Luzon 1911 (PPCS-TM/ Luzon 1911) and the Philippine Plane Coordinate System-Transverse Mercator/ Philippine Reference System of 1992 (PPCS-TM/ PRS92).

The local plane coordinate system used floating or local origin of the reference or tie-point, usually a Bureau of Lands Location Monument (BLLM) of a cadastral project. All cadastral surveys and isolated surveys done before 1965 used this coordinate system. Directions were obtained using astronomic observations. However, connecting cadastral survey outputs of various municipalities became a problem. PPCS-TM/Luzon 1911 was established to provide a unified coordinate system in the Philippines that allowed the connection of different cadastral and isolated surveys. This was established and realized from the combined astronomic and triangulation surveys in the country. Pursuant to Land Circular no. 64 dated 30th of June 1965, this system was adopted as the Philippine coordinate system (DENR, 2007). PPCS-TM/Luzon 1911 provided reference points with geographic and grid positions based on the Clarke Spheroid of 1866 parameters and the Transverse Mercator projection. PPCS-TM/ PRS92 is the current reference system used in the Philippines through Executive Order (EO) 45, series of 1993, as amended. It was a modified version of PPCS-TM/Luzon 1911 and established through GPS surveys all over the country. Both Luzon 1911 and PRS92 datum considered Station Balanacan as the origin.

PRS92 was linked to the previous Luzon 1911 datum by the connection of recovered triangulation stations through the use of GPS as part of the initiative to modernize the reference coordinate system of the country that allowed the use of satellite positioning technology for surveys and mapping purposes. Through the Natural Resources Management and Development Project (NRMDP) in 1989 to 1992, seven (7) transformation parameters were derived to transform the World Geodetic System of 1984 (WGS84) coordinates to PRS92 datum to using the old triangulation stations. WGS84 is the standard datum used by GPS. The transformation parameters from WGS84 to PRS92 are as follows: DX=+127.62195m, DY=+67.24478m, DZ=+47.04305m, Rx=-3.06762, Ry=+4.90291, Rz=+1.57790 and scale=1.06002. However, recovered old triangulation stations were non-homogeneous (NRMDP, 1990). This resulted to a non-perfect fit of the old triangulation stations to PRS92 and a significant difference between the coordinates of points using Luzon 1911 and observed coordinates using GPS in the PRS92 datum.

1.2.2 Common Point

Section 127 and 128 of DMC 2010-13 stipulates the use of a common point from previously approved survey as reference for subsequent surveys other than tie-points. Common point is generally defined as a monumented corner of a surveyed lot that defines the relative position between two or more adjoining lots (Fernandez, 1966). Recovered survey lot corners can be considered as common points if these are physically stable, fixed, undisturbed and with high degree of positional accuracy. Positional accuracy is assessed by getting the common difference between the theoretical coordinates of the recovered corners and the observed coordinates of the corners. The acceptable difference set by the Manual for Surveys is less than or equal to ±10 centimeters (DENR, 2007). However, prior to DAO 2007-29, positional accuracy limit were ±10cm for residential and ±30cm for agricultural areas (DENR, 1998). At least three monumented lot corners are needed to obtain a good common point (Fernandez, 1966). When the position of the recovered corner is within the allowable positional accuracy, the coordinates of the said corner can be used to provide positions for subsequent surveys (DENR, 2010). Common points are especially helpful if the original tie-point is distant from an isolated lot or the said tie-point was disturbed or lost.

However, the problem with the use of common point is that small amount of translation in the common point will result to an equivalent shift in the positions of the subsequent surveys and the errors become cumulative when using different common points from different surveys (Fernandez, 1966).

1.2.3 Azimuths and Distances

The azimuth of a line is the direction given by the angle between the meridian and the line measured in a clockwise direction either from the north or south branch of the meridian (Anderson & Mikhail, 1998). Three (3) types of azimuths are used in land surveying including astronomic, geodetic, and grid azimuth. The “astronomic azimuth” or “true azimuth” is the direction given by the horizontal angle between the astronomic meridian of the observer and the line measured in a clockwise direction either from the north or south of the astronomic meridian. The “geodetic azimuth” is the direction given by the horizontal angle between the ellipsoidal meridian of the observer and the line measured in a clockwise direction from the north or south of the ellipsoidal meridian. It can be obtained through differential GNSS observation. The process includes transformation from WGS84 to PRS92. Thus producing the geodetic latitude and longitude based from PRS92. To obtain the geodetic azimuth from the geodetic latitude and longitude, the Gauss Mid-Latitude Method can be used (Rapp, 1979). The "grid azimuth" is the direction given by the horizontal angle between the grid meridian (a meridian made parallel to the central meridian of a plane rectangular system) of the observer and the line measured in a clockwise direction from the north or south of the grid meridian (Wolfgang, 1980). Distortion in the angular direction can be observed as part of the effect of map projection. Grid azimuth can be obtained from the computed grid coordinates based on the projection system used.

Geodetic azimuth and astronomic azimuth are different from each other in the geodetic level but practically equal for lower accuracy requirement such as for project control use (Dimal & Balicanta, 2009). Geodetic accuracy requirement requires the conversion from geodetic azimuth to astronomic azimuth. Conversion can be done using the Laplace Correction (Leick, 2004).Grid azimuth is related to geodetic azimuth for short baselines of less than 1.5 kilometers by the convergence correction.

In the practice of land surveying in the Philippines prior to PRS92, astronomic azimuths were used to provide directions in the establishment of geodetic networks for triangulation, cadastral and isolated land surveys. With the availability of GNSS technology, grid azimuth from grid coordinates of PPCS-TM/PRS92 became readily obtainable. The Manual for Surveys provides specifications for azimuth accuracy and precision assessment for control baseline direction categorized as primary, secondary and tertiary accuracy wherein the probable error should be less than or equal to 5 seconds, 10 seconds and 15 seconds respectively. However, in practice, directions provided in the technical description of a lot are rounded-off to the nearest angular minutes.

There are basically three distances involved relating actual ground survey and mapping. These are ground distance, geodetic distance and grid distance. Ground distance is the actual distance measured between two points on the ground. Horizontal distances measured using tape or electronic distance measuring devices are considered ground distance. When corrected to the mean seal level or reference ellipsoid ground distances are reduced to geodetic distance. The effect of projection then transforms geodetic distance to grid distance. Distances provided in the technical description of a lot are rounded-off to the nearest centimeter.

1.2.4 Global Navigation Satellite System

The Philippines is in the region wherein most of the existing satellite positioning systems are available. Currently four (4) GNS Systems are available in the country. These are GPS, GLONASS, BEIDOU and QZSS. GPS or the Navigation Satellite Timing and Ranging Global Positioning System (NAVSTAR-GPS) was developed by the United States was the first fully functional satellite system used for military and commercial uses. GLONASS or GLObal'naya Navigatsionnaya Sputnikovaya Sistema was developed and managed by the Russian Space Forces for the Russian Federation Government. BEIDOU or Beidou Satellite Navigation System (BDS) formerly COMPASS was developed by the

Republic of China. Quazi Zenith Satellite System (QZSS) is maintained and operated by Japan Aerospace Exploration Agency (JAXA).

Differential technique is the conventional method used to increase the accuracy of GNSS observations. Static and rapid static differential methods are established methodologies used for geodetic and project control establishment. These require receivers to be placed on a point for several minutes and epoch rate is usually set at 15 seconds. Table 1 shows the duration for a static and rapid static surveys. The output of the process requires post processing to determine the corrected and adjusted positions of the unknown points.

Table 1. Typical Session Lengths for Static and Rapid Static Surveys (Ghilani & Wolf, 2008)

Method of Survey	Single Frequency	Dual Frequency
Static	30 min + 3min/km	20 min + 2min/km
Rapid Static	20 min + 2min/km	10 min + 1min/km

RTK-GNSS survey is a kinematic technique that provides instantaneous positioning through differential technique and transmission of the base receiver's position and raw GNSS observables to the roving receiver through radio link. Dual frequency GNSS receivers are used in conducting RTK-GNSS surveys. The epoch rate is usually set at 1 second to account for the short occupation time of the rover. Positional accuracy is in the range of $\pm$ (1cm to 2cm +2ppm) (Ghilani & Wolf, 2008). The established applications of RTK are in topographic and as-built surveys.

GNSS performance is limited by factors that affect the transmission and receiving of signals from the satellite vehicles to the GNSS receivers. These include obstructions, multi-path and noise. These should be considered in for choosing the location of observation. The final output of any GNSS techniques are coordinates in terms of either geographic coordinates or grid coordinates. Transformation from one datum to another (e.g. from WGS84 to PRS92) maybe needed to relate GNSS outputs to local datum.

1.2.5 Comparison between GPS derived and Astronomic Azimuths

The need to study the similarities and differences between GNSS derived azimuth and astronomic azimuth is important since old surveys made use of the latter. Chang and Tsai achieved a difference of $\pm$1 second between static GNSS solution and astronomic azimuth and $\pm$9 seconds for kinematic mode (Chang & Tsai, 2006). A similar experiment was conducted by the U.P. Training Center for Applied Geodesy and Photogrammetry (UPTCAGP) for the Research and Development Project in Support of the Implementation of the Philippine Reference System of 1992 funded by the National Mapping and Resource Information Authority (NAMRIA). The study showed that using rapid static survey, geodetic azimuths computed from logged coordinates and published coordinates of NAMRIA reference points are comparable at least within secondary accuracy (UPTCAGP, 2009). The research showed that for project control, geodetic azimuth and astronomic azimuth can be used interchangeably within secondary accuracy.

1.2.6 RTK-GNSS Performance Under Tree Canopy

Small openings for areas under tree canopies allow GNSS satellite signals to pass through and be received by GNSS receivers. The experiment conducted by Lucas in 2007 showed that the best accuracy achieved by survey grade GPS receiver is 1.46 meters using post processed kinematic technique (Lucas, GPS under the Forest Canopy, 2007). Zheng J. et. al made use of RTK and related it to canopy index derived from photographs underneath the canopy and applying Otsu's method through image processing. The relationship between positional dilution of precision (PDOP) and canopy density index was shown. Results show that positional accuracy is degraded as the density of the canopy increases and that positional update is delayed (Zheng, Wang, & Nihan, n.d.).

1.2.7 LMB Memorandum Circular No. 2015-001

DENR-LMB issued Memorandum Circular No. 2015-001 in 14th January 2015. It is a policy guideline on the use of RTK for all kinds of lots surveys with tertiary accuracy. The guideline includes procedures and conditions that must be considered when using RTK-GNSS survey. Salient conditions are (a) use of calibrated and tested dual frequency GNSS receivers, (b) receiver clearance of 15^{o} from the horizon, (c) bipod support of poles with receivers during survey, (d) use of electronic total station to augment RTK-GNSS, (e) root mean square (RMS) value must be 35 or below, (f) RTK observation length not less than two (2) minutes,(g) minimum of five (5) satellites must be tracked and (h) baseline length of 200 meters to 1 kilometer shall be established using RTK instrument preferably not more than 1 kilometer from the lot (DENR-LMB, 2015). In addition, a list of required contents of observation field notes was provided.

2. METHODOLOGY

Figure 1 shows the workflow of the experiments conducted in the research to assess the applicability of RTK-GNSS for property survey purposes. These include Experiment 1: test on the azimuth obtained from RTK-GNSS survey, Experiment 2: use of RTK-GNSS under tree canopy, Experiment 3: Use of RTK-GNSS for new survey and Experiment 4: Use of RTK-GNSS for old survey. Several locations were selected as study area in the conduct of the four experiments. Different dual frequency GNSS receiver models were used depending on the purpose and availability of the equipment. Familiarization of the different equipment was done prior to actual implementation of the experiments. Sites were selected based on accessibility and applicability to the experiments.

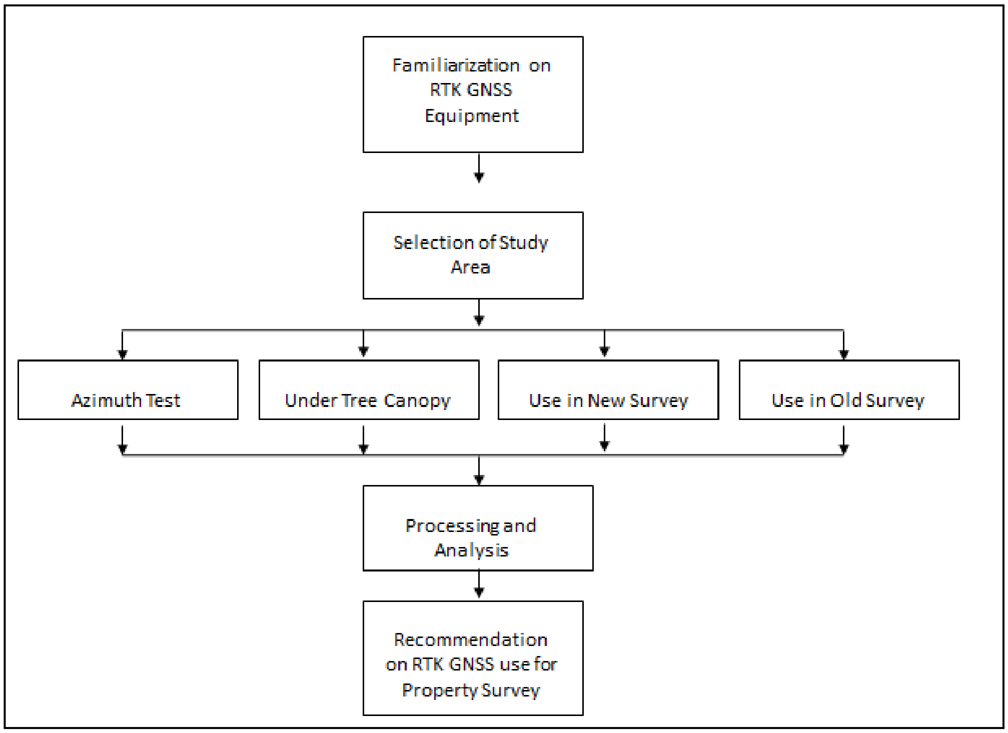

Figure 1. Workflow of the Experiments

2.1 Azimuths from RTK-GNSS

This experiment was conducted to compare the azimuths derived from RTK-GNSS and rapid-static GNSS techniques. Experiments were performed at Barangay Sta. Cruz, Guiguinto, Bulacan (Site 1) and at the calibration baseline within the U.P. Diliman Campus, Quezon City (Site 2).

Figure 2. Site 1 used for Azimuth Test

Figure 3. Site 2 used for Azimuth Test

Two (2) EPOCH 50 Dual Frequency receivers were used in the experiment. This receiver model is capable of receiving GPS and GLONASS satellite signals. Three points were selected and marked on the ground using concrete nail and named STN1, STN2 and CMA1. STN1 was used as base-station although not a NAMRIA established reference control point.

Rapid static survey was conducted and set at 30 minute observation. Baseline STN1-STN2, STN1-CMA1 and MMA39-MMA39B2 were observed. Post processing was done to obtain the position of the three (3) points. The resulting WGS84 positions of the points were transformed to PRS92 using the NRMDP transformation parameters, then the PRS92 geographic positions were projected to PPCS-TM/PRS92.

RTK-GNSS was conducted having the base receiver set at STN1 and rover receiver at STN2 and CMA1. Base receiver was set on a bipod to provide stability of the base station. Roving receiver was held by hand to test if this is applicable for the purpose. Epoch interval set was 1 second with a minimum averaging of 6 for fixed solutions. Twenty (20) averaged position solutions were done for STN2, CMA1 and MMA39B2.

Plotting of points was done using a third party computer aided design and drafting (CADD) software to show the positions from the RTK-GNSS relative to the position obtained through Rapid Static GNSS Survey. Grid and geodetic azimuths were computed. A comparison between derived azimuths from rapid static GNSS and RTK-GNSS was done. Differences in the results were assessed using azimuth accuracy requirement as per Section 28 of DAO 2007-29.

2.2 RTK-GNSS under Tree Canopy

This experiment was conducted to test the performance of different RTK-GNSS models under tree canopy and was performed together with several students of GE 117: Construction Surveying. The GNSS receivers used in the experiment include CHC X900+GNSS receiver capable of receiving satellite signals from GPS, GLONASS and BDS and Trimble SPS 882 capable of receiving satellite signals from GPS and GLONASS. The research experiment was conducted within the premises of the University of the Philippines Diliman Campus specifically at the north-western portion of the academic oval (Site 3) near Quezon Hall. Thirty (30) points were marked on selected areas about 30 to 50cm from the trunks of selected trees. The location of each point was chosen based on visual distinction of tree cover. CHC X900+GNSS was used from one hour and twenty minutes followed by the use Trimble SPS 882 for one hour and forty one minutes.

Both GNSS receivers were tested using the RTK feature of the equipment. Base receivers were set-up at MMA-39 using its WGS84 geographic coordinates N 14º 39’ 18.82950”, E 121º 03’ 34.94441” obtained from NAMRIA. After initialization, rover receivers were set-up at the selected 30 test points. One (1) second epoch interval was set for both GNSS models. Three (3) minutes was set as limit and “Float” or “Autonomous” solutions beyond the time limit set was noted as “NOT FIXED”. Satellite constellations including number and type of satellite, PDOP, XYZ precision and position from the controllers were noted during the observation. Results were tabulated and compared.

Figure 4. Site 3 for RTK-GNSS under Tree Canopy Experiment

2.3 RTK-GNSS for New Survey

This experiment was conducted to test RTK-GNSS in the conduct of a new parcel survey referred from a known point. The experiment was done near the calibration baseline within U.P. Diliman, Quezon City (Site 4).

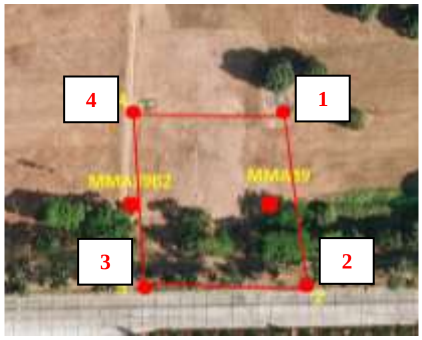

Figure 5. Site 4 for RTK-GNSS for New Survey Experiment

Two (2) EPOCH 50 Dual Frequency receivers capable of obtaining GPS and GLONASS signals and a NIKON 1-second reading total station were used in the experiment. Four (4) points were selected to simulate the boundary corners of a lot. Corners were marked on the ground and denoted as 1, 2, 3 and 4. A bipod was used to stabilize the base-station while rover receiver was held by hand.

RTK-GNSS was performed at these points using MMA39 as the base station and the grid coordinates used were 1620828.721 meters Northing, 506285.077 meters Easting. A minimum averaging of 6 observations was set for fixed solutions. Five (5) averaged position solutions were done to the simulated four corners. A total Station was then used to measure the azimuths and horizontal distances from MMA-39 to the four corners having the computed geodetic azimuth of the calibration baseline as the initial direction.

Plotting of points was done using third party CADD software to show the positions from RTK-GNSS relative to the position obtained through measurements using total station. In addition to grid and geodetic azimuth, grid and geodetic distances were computed. The technical descriptions (in terms of bearing and distance) were computed in the grid and geodetic systems. These were compared from the results of total station-derived technical descriptions.

2.4 RTK-GNSS for Old Survey

The experiment was done to test the performance of RTK-GNSS for evaluating positions of boundary monuments from a previously surveyed lot and using common point. The fieldwork was done together with students from GE 174: Satellite Positioning System. A Tripod was used at the base-station while the rover receiver was held by hand.

The GNSS receivers used in the experiment were Hi-Target V30/50 RTK GNSS receivers with only GPS and GLONASS available at the time. Prior to actual ground survey, an approved survey plan was researched. Consolidation and Subdivision plan, Pcs-03-015395 was obtained from DENR Land Management Service (LMS) Region III containing four (4) lots. The research team asked permission to the land-owner to access the lots. The study area is located at Barrio Tabe, Guiguinto, Bulacan (Site 5). The technical descriptions of the subject lots were plotted using third party CADD software. Upon reconnaissance, the team observed that Lots 1 and 2 had complete boundary monuments. Lot 1 has six (6) corners and Lot 2 has four (4) corners. Two points are common to the two lots. The eight (8) points have minimal obstruction.

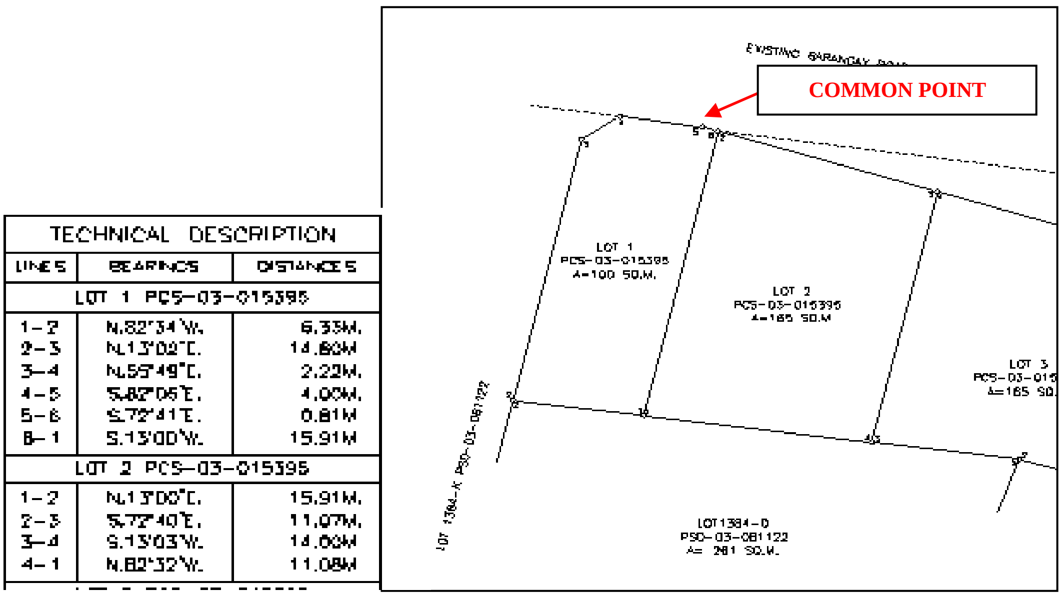

Figure 6. Survey plan with technical description of Site 5 used for RTK-GNSS for Old Survey Experiment

RTK-GNSS survey was performed using an arbitrary point as base-station since the reference point/ tie-point was not recovered. The rover receiver was used on the boundary monuments and obtained the coordinates of each point tied to the position of the base-station. The initial outputs were then overlaid on the CADD drawing of the subject lots. Positions of the drawing entities such as points and lines were translated from the coordinates provided by RTK-GNSS to the coordinates of the points based on the tie-point/ tie-line. Since no actual reference point was occupied, common point analysis was performed to choose from the eight (8) points a point that would provide differences in position less than or equal to 10 centimeters.

3. RESULTS AND DISCUSSIONS

3.1 Azimuths from RTK-GNSS

Figures 2 and 3 show the location of the established points for azimuth derivation using RTK-GNSS. STN2 can be described as obstructed as it was located near structures and tall trees and CMA1 is located on a clear area. MMA39B2 is located on an area with tall trees south of its location. Table 2 shows the mean positional accuracy, number of satellites and maximum PDOP during the RTK-GNSS experiment. As expected STN2 was observed to have lower horizontal precision and high PDOP because of its location, however all three points have horizontal precision better than $\pm$2cm. The maximum deflection and standard deviations in the northing and easting directions are provided in Table 3. The tables show that differences in position between the output of static survey and RTK reach a maximum of 3cm for STN2 and CMA1 and 7cm for MMA39B2. Table 4 shows the computed grid azimuths of the baselines using the coordinates from static survey compared to the grid azimuths computed from the coordinates from RTK survey with the maximum angular difference to static position. The table shows that angular difference between static and RTK survey results can be of large magnitude and probable error can greater than the allowable 15 seconds for tertiary accuracy as specified by the Manual for Surveys. Table 5 shows the comparison of grid azimuths of baseline MMA39-MMA39B2 obtained using MMA39 NAMRIA grid coordinates and using the logged grid coordinates of MMA39. The results reiterated the conclusion of UPTCAGP that practically the same azimuths can be obtained using either NAMRIA published coordinates of a reference station or its logged position. Table 6 shows the comparison between the computed geodetic azimuth of MMA39 using static survey with NAMRIA coordinates and the astronomic azimuth of the baseline from UPTCAGP research equal to 89^{o} 45’ 45” (UPTCAGP, 2009).

Table 2. Mean horizontal position, minimum no. of satellites and maximum PDOP of the four simulated corners

PT. ID	mean Horizontal Precision (m)	mean minimum no. of satellites	mean Maximum PDOP
STN2	0.017	10	2.795
CMA1	0.011	10	1.815
MMA39B2	0.014	11	2.082

Table 3. Maximum Difference and Standard Deviations between Rapid-Static GNSS positions and RTK GNSS positions

Point ID	Δ Nmax	Δ Emax	σ N	σ E
STN2	-2.5cm	-2.1cm	±1.3cm	±1.1cm
CMA1	1.8cm	-2.9cm	±1.4cm	±0.8cm
MMA39B2	-3.4cm	-6.6cm	±3.6cm	±1.8cm

Table 4. Comparison of Grid Azimuths from Static and RTK Survey

LINE	Grid Azimuth						Difference			PE<15 sec
	From Static			From RTK						Yes/ No
	dd	mm	ss	dd	mm	ss	dd	mm	ss	
STN1-STN2	339	14	30.99	339	18	34	0	-4	-3.01	No
STN1-CMA1	72	20	33.39	72	20	22	0	0	11.39	Yes
MMA39-MMA39B2	89	45	0.69	89	40	28	0	4	32.69	No

Table 5. Comparison of Grid Azimuths using Namria Position and Logged Position of MMA39

LINE	Grid Azimuth						Difference			PE<5 sec
	NAMRIA Position of MMA39			Logged Position of MMA39						Yes/ No
	dd	mm	ss	dd	mm	ss	dd	mm	ss	
MMA39-MMA39B2	89	45	0.69	89	45	0.51	0	0	0.18	Yes

Table 6. Comparison of Geodetic Azimuth and Astronomic Azimuth

LINE	Geodetic Azimuth (Grid Azimuth w convergence correction			Astronomic Azi muth (UPTCAGP 2009)			Difference			PE<10 sec
										Yes/ No
	dd	mm	ss	dd	mm	ss	dd	mm	ss	
MMA39-MMA39B2	89	45	53.84	89	45	45	0	0	8.84	Yes

The result shows that the difference between astronomic and geodetic azimuths is negligible and meets the secondary accuracy requirement of the Manual for Surveys. These experiments show that fixed solutions tend to depend on good horizontal precision and positional dilution of precision rather than the actual number of satellites used. For property survey purposes, horizontal precision of three (3) centimeters or less and PDOP better than 3 are recommended to be used as indicators of a good observation. The difference between positions from Rapid-Static GNSS and RTK-GNSS are within the centimeter level on an ideal setting. However such differences may result to large errors in the azimuths produced by RTK-GNSS that exceed the allowable accuracy requirement for azimuth determination. Averaging of six (6) 1-second RTK-GNSS solution obtained centimeter level accuracy. Increasing the number of observations may increase solutions and the confidence in the result.

3.2 RTK GNSS under Tree Canopy

Table 7 shows the list of the points which were fixed based on the three (3) minute rule. From the result it was observed that 53% of the points were not fixed within three (3) minutes for GPS+GLONASS only receiver. Fastest fixing time was 19 seconds. 100% of the points were fixed within three (3) minutes for GPS+GLONASS+BDS receiver. Fixing time ranges from 3 seconds to 2 minutes and 17.07 seconds. The number of observed satellites from GPS+GLONASS+BDS receiver was observed to be always higher than its counterpart. However, it must be noted that the two receivers were used at different times of the day.

Table 8 shows the obtained position of points which were fixed in both receivers. Difference ranges from 0.1 m to 7.5 m for northing and 0.8m to 5.9m for easting. Validation survey to provide a check data for the positions of the thirty selected points was not done.

Table 7. List of points observed to be fixed and not fixed.

Tree no.	GPS+GLONASS Receiver (Date: 10/22/2013, Time: 16:03-17:44)		GPS+GLONASS+BDS Receiver (Date: 10/22/2013, Time: 11:29-12:49)	
	No. of Sat	Fixed/ Not Fixed	No. of Sat	Fixed/ Not Fixed
1	9	Not Fixed	10	Fixed
2	7	Fixed	22	Fixed
3	8	Fixed	14	Fixed
4	11	Fixed	15	Fixed
5	11	Not Fixed	17	Fixed
6	10	Not Fixed	21	Fixed
7	12	Fixed	21	Fixed
8	10	Fixed	15	Fixed
9	12	Fixed	15	Fixed
10	11	Fixed	15	Fixed
11	11	Not Fixed	14	Fixed
12	9	Not Fixed	16	Fixed
13	5	Not Fixed	14	Fixed
14	10	Not Fixed	16	Fixed
15	8	Not Fixed	18	Fixed
16	8	Not Fixed	16	Fixed
17	8	Fixed	20	Fixed
18	9	Fixed	16	Fixed
19	9	Not Fixed	19	Fixed
20	8	Fixed	13	Fixed
21	9	Not Fixed	14	Fixed
22	10	Not Fixed	10	Fixed
23	10	Not Fixed	15	Fixed
24	10	Not Fixed	13	Fixed
25	13	Fixed	16	Fixed
26	8	Fixed	15	Fixed
27	8	Not Fixed	13	Fixed
28	10	Not Fixed	18	Fixed
29	7	Fixed	13	Fixed
30	9	Fixed	13	Fixed

Table 8. WGS84 UTM Zone 51 grid coordinates (in meters) of the points

(GPS+GLONASS Receiver) (Date 10.22.2013)			(GPS+GLONASS+BDS Receiver) (Date 10.22.2013)		Comparison	
WGS84 ZONE 51						
Tree	Northings	Eastings	Northings	Eastings	Difference in Northings	Difference in Eastings
2	1621081.953	291558.618	1621086.319	291555.541	4.366	3.077
3	1621091.888	291562.315	1621095.425	291560.731	3.537	1.584
4	1621110.936	291573.44	1621115.286	291570.399	4.35	3.041
7	1621164.443	291614.008	1621168.792	291610.9	4.349	3.108
8	1621162.896	291641.73	1621168.227	291639.993	5.331	1.737
9	1621164.415	291653.258	1621168.754	291650.236	4.339	3.022
10	1621163.606	291681.486	1621167.983	291678.46	4.377	3.026
17	1621156.186	291733.925	1621160.555	291730.823	4.369	3.102
18	1621162.928	291737.825	1621167.443	291734.633	4.515	3.192
20	1621163.733	291761.85	1621171.253	291755.879	7.52	5.971
25	1621164.087	291802.822	1621168.459	291799.693	4.372	3.129
26	1621163.394	291811.57	1621167.815	291808.439	4.421	3.131
29	1621163.089	291851.503	1621163.077	291850.535	0.012	0.968
30	1621163.868	291858.984	1621164.091	291859.812	0.223	0.828

3.3 RTK for New Surveys

Figure 5 shows the location of the corners of the sample lot. Corners 1, 2 and 3 are located in an area with a significant number of tall trees at the immediate vicinity. Corner 4 is located in a clear area. Table 9 shows the mean horizontal precision, mean minimum number of satellites and maximum PDOP observed during the RTK-GNSS experiment. As expected, the survey on corner 4 provided good positional accuracy.

Table 10 shows the computed mean coordinates and standard deviations of the four (4) points. Table 11 shows the computed grid azimuths, grid distances, geodetic azimuths and geodetic distances from the coordinates of the 4 points reckoned from MMA39. There is a difference of around 1 minute between the grid azimuths and geodetic azimuths and up to the nearest one centimeter between the grid distances and geodetic distances. Table 12 shows the RTK GNSS results versus results from total station observations. Reduced geodetic distances of the total station observation were computed. A maximum difference of 1 minute and 12 seconds was observed comparing total station results and RTK-GNSS results for line MMA39 to corner 3. A difference of a maximum difference of around 1.1cm was observed for line MMA39 to corner 3. Table 13 shows the derived technical descriptions from grid coordinates, geodetic coordinates and total station observations of the simulated lot. There was an observed difference of 1 to 2 angular minutes between the bearings of RTK-GNSS: Grid, RTK-GNSS: Geodetic and total station observations. Distance differences are mainly in the millimeter level except for line 3-4 wherein a difference of 1.9 centimeters was obtained between the result from total station and from RTK-GNSS: Geodetic .

Table 9. Mean horizontal position, minimum no. of satellites and maximum PDOP of the four simulated corners

PT. ID	mean Horizontal Precision (m)	mean minimum no. of satellites	mean Maximum PDOP
1	0.015	11	1.993
2	0.011	11	1.696
3	0.016	13	1.696
4	0.008	13	1.702

Table 10. Mean positions (in meters) of the four (4) corners of the lot

Corners	Mean N	Mean E	σ N	σ E
1	1620860.305	506293.775	±0.2cm	±0.5cm
2	1620795.197	506298.7978	±0.3cm	±0.3cm
3	1620794.047	506238.6972	±0.2cm	±0.3cm
4	1620861.588	506236.8578	±0.2cm	±0.4cm

Table 11. Grid azimuths and grid distances versus geodetic azimuths and geodetic distances

LINE	Grid Azimuth derived from RTK GNSS			Grid Distance from RTK GNSS (m)	Geodetic Azimuth derived from RTK GNSS			Geodetic Distance from RTK GNSS (m)
	dd	mm	ss		dd	mm	ss	
MMA39-MMA39B2	89	45	0.69	49.991	89	45	53.42	50.003
MMA39-1	195	23	49.78	32.760	195	24	43	32.761
MMA39-2	337	44	30.03	36.223	337	45	22.86	36.225
MMA39-3	53	13	4.81	57.908	53	13	57	57.911
MMA39-4	124	16	43.18	58.355	124	17	37.09	58.358

Table 12. RTK-GNSS results and total station observations

LINE	Geodetic Azimuth derived from RTK GNSS			Geodetic Distance from RTK GNSS (m)	Geodetic Azimuth from Total Station Observation			Ground Distance from TS Observation (m)	Reduced Geodetic Distance from TS Observation (m)
	dd	mm	ss		dd	mm	ss		
MMA39-MMA39B2	89	45	53.42	50.003	89	45	53	50.005	50.004
MMA39-1	195	24	43	32.761	195	24	35	32.761	32.760
MMA39-2	337	45	22.86	36.225	337	46	25	36.217	36.216
MMA39-3	53	13	57	57.911	53	15	4	57.902	57.900
MMA39-4	124	17	37.09	58.358	124	17	59	58.351	58.349

Table 13. Output technical descriptions from RTK-GNSS and Total Station Surveys

TECHNICAL DESCRIPTION		
FROM GRID		
LINE	BEARING	DISTANCE (m)
1-2	S 04-24-40.932 E	65.302
2-3	S 88-54-15.063 W	60.112
3-4	N 1-33-36.042 W	67.565
4-1	S 88-42-32.715 E	56.932
AREA	3880.3337	sqm
FROM GEODETIC		
1-2	S 4-23-48.209 E	65.304
2-3	S 88-55-06.989 W	60.114
3-4	N 1-32-43.424 W	67.569
4-1	S 88-41-37.191 E	56.934
AREA	3880.702	sqm
FROM TOTAL STATION		
LINE	BEARING	DISTANCE (m)
1-2	S 4-23-11.433 E	65.301
2-3	S 88-56-05.346 W	60.105
3-4	N 1-32-04.349 W	67.550
4-1	S 88-41-33.285 E	56.924
AREA	3879.373	sqm

TECHNICAL DESCRIPTION		
FROM GRID		
LINE	BEARING	DISTANCE (m)
1-2	S 04-25 E	65.30
2-3	S 88-54 W	60.11
3-4	N 1-34 W	67.57
4-1	S 88-43 E	56.93
AREA	3880	sqm
FROM GEODETIC		
1-2	S 4-24 E	65.30
2-3	S 88-55 W	60.11
3-4	N 1-33 W	67.57
4-1	S 88-42 E	56.93
AREA	3881	sqm
FROM TOTAL STATION		
LINE	BEARING	DISTANCE (m)
1-2	S 4-23 E	65.30
2-3	S 88-56 W	60.11
3-4	N 1-32 W	67.55
4-1	S 88-42 E	56.92
AREA	3879	sqm

3.4 RTK-GNSS used for Old Surveys

RTK survey was successful and provided positions to the monument markers based on the arbitrary position of the base-point. Common point analysis was performed and the coordinates of all the points were recomputed based on the theoretical coordinates of the boundary monument considered as the common point. The common point chosen was corner 5 of Lot 1. Table 14 shows the comparison of the theoretical coordinates of the boundary monuments versus the positions of the same set of monuments obtained from the RTK-GNSS method. A second computation of the coordinates was done using the geodetic azimuth and geodetic distance of lines from the base-point to each corner. The resulting coordinates were almost equal to the results shown in Table 14 within one centimeter.

Table 14: Comparison between Theoretical Coordinates and RTK-GNSS Results (in meters)

Lot no./ Corner no.	THEORETICAL COORDINATES		RTK-GNSS RECOMPUTED COORDINATES		Δ N	Δ E	Displacement
LOT 1	NORTHINGS	EASTINGS	NORTHINGS	EASTINGS			
1	1641268.701	487923.976	1641268.729	487924.053	-0.028	-0.077	0.08
2	1641269.521	487917.696	1641269.540	487917.764	-0.019	-0.068	0.07
3	1641283.741	487920.986	1641283.664	487920.972	0.077	0.013	0.08
4	1641284.991	487922.826	1641284.899	487922.785	0.092	0.041	0.10
5	1641284.441	487926.786	1641284.441	487926.786	0.000	0.000	0.00
6	1641284.201	487927.556	1641284.190	487927.517	0.011	0.038	0.04
LOT 2							
1	1641268.701	487923.976	1641268.729	487924.053	-0.028	-0.077	0.08
2	1641284.201	487927.556	1641284.190	487927.517	0.011	0.038	0.04
3	1641280.901	487938.126	1641280.907	487938.078	-0.006	0.048	0.05
4	1641267.261	487934.966	1641267.281	487934.988	-0.020	-0.022	0.03

3.5 Combined RTK-GNSS-Total Station Methodology

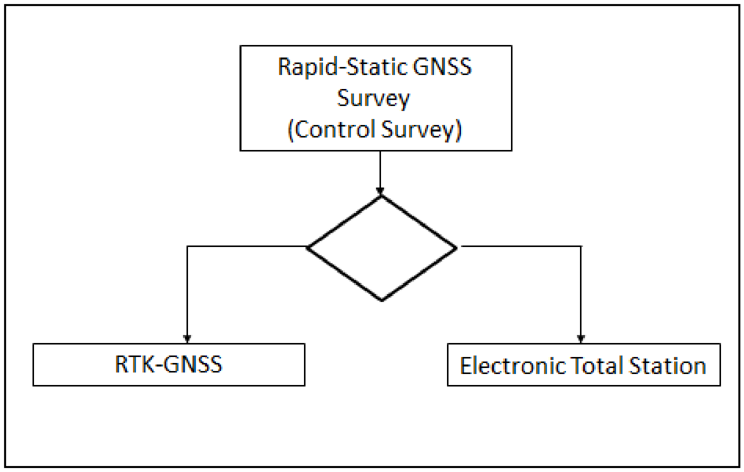

Figure 7. Combined RTK-GNSS-Total Station Methodology

The results in the previous sections showed that outputs of RTK-GNSS Survey and electronic total station in the survey are comparable. A combined RTK-GNSS-Total Station Methodology is proposed to provide land surveyors procedures and guidelines on applicability of RTK-GNSS in the conduct of any property surveys.

Rapid-Static GNSS Survey can be used in establishing controls for the survey. Control points can be connected to old Bureau of Lands reference stations for property surveys dealing with old surveys or to new established control points from NAMRIA or DENR-LMS depending on the situation. Established control points should at least be a pair to provide consideration to the use of electronic total station. Baseline length can be short but at least 50 meters is suggested.

The use of RTK-GNSS or electronic total station is interchangeable and a surveyor must decide whether to use one over the other. RTK-GNSS Survey can be used based on the conditions set by DENR-LMB Memorandum Circular No. 2015-001. However based on this research, some improvements on the conditions are provided. The modified and additional conditions are (a) receiver clearance of 15^o from the horizon can be waived if GNSS receiver can provide positional error not more than five (5) centimeters under satellite-signal obstructions such as tree canopies, (b) rover receivers held is applicable if the circular bubble is maintained to be balanced and stable by the instrument man, (c) positional precision should be better than five (5) centimeters can be used as guide to a good observation if RMS is not provided, (d) observation time can be lessened to five (5) 1-second observations depending on the precision shown on the GNSS controller, (e) minimum of 10 satellites are needed to have a good RTK-GNSS Survey results, and (f) PDOP should be better than 2. If these conditions are met, RTK-GNSS can be used for a specific property survey task. If not, the traditional method of using an electronic station should be used. Grid azimuths and grid distances must be converted to geodetic azimuths for total stations

4. CONCLUSIONS AND RECOMMENDATIONS

The research study successfully tested and evaluated the use of RTK-GNSS for property surveys. RTK-GNSS was shown to be applicable in determining the positions of parcel corners without significant difference compared to the traditional method, whether it is a new survey or an old survey. Methodologies provided in the research may be used by survey practitioners in conducting property surveys using RTK-GNSS stand-alone or RTK-GNSS combined with an electronic total station. However, it has limitation such as obstruction, accuracy within centimeter level and systematic error from projection. RTK-GNSS like the traditional method is also affected by land survey conditions such as varying coordinate system and poor or wrong identification of common point.

Due to obstructions which cannot be avoided especially in urban areas, a combined RTK-GNSS-total station survey is the preferred technique. In combination with an electronic total station survey, Rapid-Static GNSS Survey is recommended in azimuth determination rather than RTK-GNSS. The difference between grid and geodetic azimuths must be considered in combining GNSS and total station surveys. Since the main output of RTK-GNSS survey is in terms of grid position there maybe a need to change the requirements for survey plan approval since traverse and lot data computations using side-shot data are not applicable. It is recommended that part of the submittal information include base-point used, horizontal precision obtained during the survey and PDOP. Solutions on getting the geodetic azimuth and distance may also be included since survey plans are currently in terms of directions and distances. Furthermore, the requirement to provide only grid positions only can also be recommended. Finally, problems pertaining to the three (3) existing coordinate reference systems in the current cadastral database and the possible difference between the technical description of an old survey and the result of RTK-GNSS on lot parcels were not covered by the experiments. Surveyors must be aware of these issues when doing surveys to be able to adjust to the situation and provide a sensible solution to a particular property survey problem. These are also topics for future research endeavors.

5. ACKNOWLEDGEMENT

The research was made possible due to the F.F. Cruz Professorial Chair Award. The author would like to acknowledge previous students Engr. Laura Melissa Chua, Engr. Diana Igos, Engr. Ma. Ailyn Olanda, Engr. Sarah Jane Samalburo, Ms. Patricia Kristen Dela Cruz, Mr. Edison Paraiso, Ms. Ziarre Anne Mariposa, Mr. Jerome Santiago, Ms. Marra Sunga, Ms. Geozza Tan and Ms. Lianne Zausa for assisting in the data gathering fieldworks. Acknowledgement is also given to UPTCAGP, RASA Surveying, MJAS Zenith Trading and the UP-DOST DREAM Program for lending their equipment, Dr. Enrico Paringit for providing some materials used in the review of related literature and Engr. Rosalia Balicanta of the Department of Environment and Natural Resources for technical discussions on topics related to the Manual for Land Surveys.

6. REFERENCES

[1] Anderson, J., & Mikhail, E. (1998). *Surveying Theory and Practice Seventh Edition.* New York: McGraw-Hill Companies, Inc.

[2] Chang, C., & Tsai, W. (2006). Evaluation of a GPS-Based Approach for Rapid and Precise Determination of Geodetic/Astronomical Azimuth. *Journal of Surveying Engineering*, 149-154.

[3] DENR. (1998). DAO 1998-12: Manual for Surveyors.

[4] DENR. (2007). *DAO 2007-29: Revised Regulations on Land Surveys.* Quezon City, Philippines: DENR.

[5] DENR. (2010, June 23). DMC 2010-13: Manual on Land Survey Procedures. Quezon City.

[6] DENR-LMB. (2015, January 14). LMB MC 2015-001" Guidelines on the Use of Real Time Kinematic (RTK) Global NAvigation Satellite System (GNSS) in the Conduct of All Kinds of Lot Surveys with Tertiary Accuracy. Manila, Philippines: DENR Land Management Bureay.

[7] Dimal, M. O., & Balicanta, L. P. (2009). Comparative Analysis of GPS Azimuth and Derived Azimuth for the Establishment of Project Control. *7th FIG Regional Conference* (p. 18). Hanoi, Vietnam: FIG.

[8] Ghilani, C. D., & Wolf, P. R. (2008). *Elementary Surveying: An Introduction to Geomatics Twelfth Edition.* New Jersey: Pearson Prentice Hall.

[9] Fernandez, J. R. (1966). Comments on the Practice of Property Delimitation Survey in the Philippines. *Philippine Journal of Geodesy and Photogrammetry*, 75-80.

[10] Leick, A. (2004). *GPS Satellite Surveying.* John Wiley and Sons.

[11] Lucas, K. (2007). *GPS under the Forest Canopy.* Retrieved October 23, 2013, from www.proaxis.com/~johnbell/guest/GPS_Canopy_Tests.pdf

[12] NRMDP. (1990). *Geodetic Survey of the Philippines Final Report.* Philippines: NRMDP.

[13] Rapp, R. H. (1979). *Geometric Geodesy*. Columbus, Ohio: Ohio State University.

[14] UPTCAGP. (2009). *Comparative Analysis and Conversion of Azimuth Obtained from GPS and Astronomic Observation.* Quezon City, Philippines: UPTCAGP-NAMRIA.

[15] Wolfgang, T. (1980). *Geodesy: An Introduction.* Berlin: Walter de Gruyter & Co.

[16] Zheng, J., Wang, Y., & Nihan, N. L. (n.d.). *Quantitative Evaluation of GPS Performance under Forest Canopy*. Retrieved October 10, 2013, from Faculty.washington.edu/yinhai/wangpublication_files/ICNSC_05_GPS.pdf