DEMARCATION OF BOUNDARIES USING SATELLITE TECHNOLOGIES: COMPARISON BETWEEN 'HIGH END GPS' AND 'LOW END GPS'

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ABSTRACT

Demarcation of boundary marks for surveying applications using the conventional surveying techniques (theodolite, EDM and total station) is tedious, time consuming, labour intensive and costly. Further more its requires inter-visibility between survey points.
With the advancement of satellite technology, positioning with high accuracy can be achieved using the Global Positioning System (GPS) technology (U.S.Army, 1996). The GPS technology is able to challenge this short range inter-visibility constraint whereby each point is observed independently. The capability of this system towards point positioning is also highly recognized in mapping application (Leick, 1995). In this study, the capability of GPS receivers towards demarcation of points is explored. Discussion will be made on the technique of survey and the capability of the equipment used. Comparison will be made on the derived bearing and distance as compare to EDM (Electronic Distance Measurement) traverse (Fadzil, 2000). The relationship between the coordinate system (Bravier, 1974 and Mailing, 1980) in Malaysia as compare to GPS system will also be discussed.

Keyword: Global Positioning System, High and Low End GPS, Coordinate system.

Introduction

GPS is a space-based radio positioning system that provides 24 hours three-dimensional position to suitably equipped users anywhere on the surface of the earth (Holfmann, 1992). The accuracy of positioning in GPS survey depends on many factors. These include, the reliability of the receiving equipment, techniques of survey & processing and satellite configuration (Kamaluddin, 2000).

At present, the accuracy of point positioning using a single commercial-of-the-shelf GPS receiver is within few meters. However, as mention earlier the accuracy of the established point can be upgraded with respect to observation techniques and method of processing (Rizos, 1996). GPS receivers can be group into; high end, low end and hand held receivers. In this study, the high and low end receivers towards point positioning will be explored. The high-end receivers utilized the L1 and L2 frequency with 1575.42MHz and 1227.60MHz respectively, where else the low end receivers is equipped with the L1 frequency.

Methodology

Figure 1 shows the position of known boundary marks for a closed traverse established using the cadastral surveying technique (Roslee, 1998). Bearing misclosed was adjusted and distributed using the Bowditch method. The accuracy of the traverse is 1: 669 566. Table 1 shows the computed value for the closed traverse.

ISSN 1675-5022

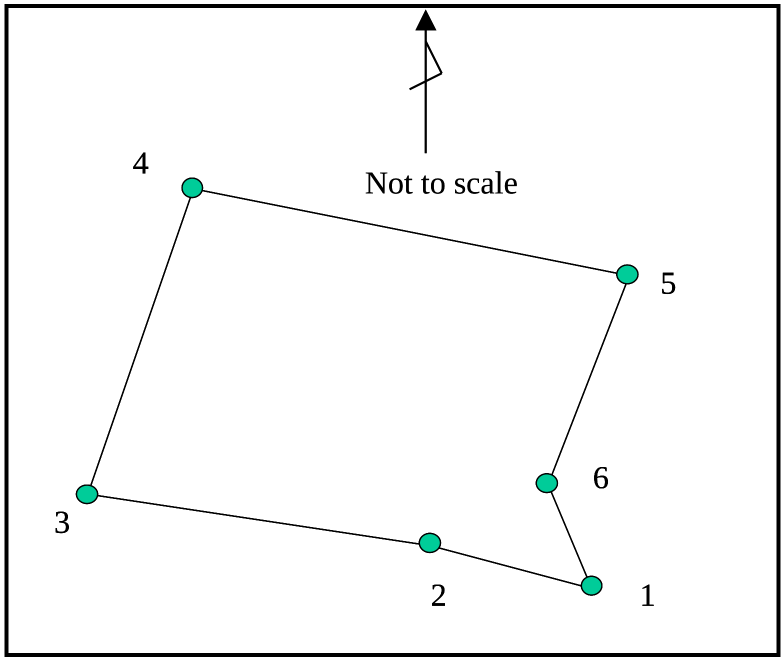

Figure 1: The closed traverse

Table 1: Bearing, distance, latitude and departure for the closed traverse.

Stn	Bearing	Distance (m)	Northing (m)	Easting (m)
1 – 2	298° 28’ 26”	130.625	+62.277	-114.824
2 – 3	300° 40’ 36”	291.634	+148.790	-250.823
3 – 4	016° 26’ 51”	285.581	+273.895	+80.858
4 – 5	108° 10’ 20”	414.975	-129.420	+394.277
5 – 6	202° 22’ 36”	224.356	-207.462	-85.411
6 – 1	189° 14’ 09”	150.024	-148.079	-24.079
		Σ1497.195	Δ 0.001	Δ 0.002

Apart from conventional surveying approach, the positions of the points are determined using GPS receivers. It should be noted that, the GPS receivers used in the campaign have undergone the EDM baseline test (JUPEM, 1999). The method of observation with respect to the high and low-end receivers is depicted in Figure 2.

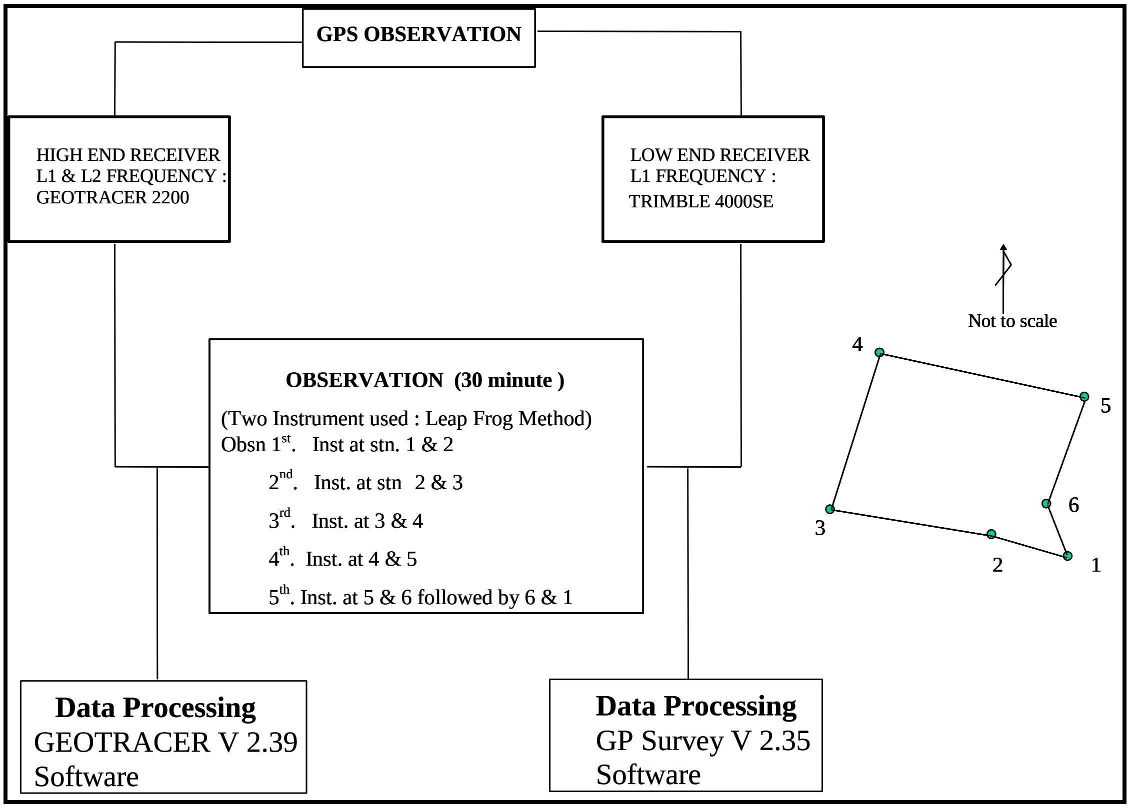

Figure 2: Observation method using GPS.

Referring to Figure 2, the points are observed using the high and low-end GPS receivers independently. The survey method adopted is by using the *leap frog* approached (Che, 1996). The processing for the gathered high and low end GPS dataset is by GEOTRACER V 2.39 and GPSurvey V2.35 respectively. The time for observation is approximately 30 minutes.

Results & Discussion

Table 2 shows the difference in derived bearing and distance between the traverse dataset and high end GPS. Referring to Table 2, it is shown that the discrepancies between the derived bearing and distance for each traverse line are less than 25" and 0.02 m respectively. Table 3 depict the accuracy for the closed traverse using the high end GPS which is 1: 748 569.

Table 2: Diff. in bearing and distance bet. traverse stations (high-end GPS observ.)

STN	**TRAVERSE**		**GEOTRACER**		**DIFFERENCE**	
From-To	Bearing	Dist(m)	Bearing	Dist(m)	Azimuth(")	Dist(m)
1 – 2	298 28 26	130.625	298 28 06	130.613	20	0.012
2 – 3	300 40 36	291.634	300 40 46	291.631	-10	0.003
3 – 4	016 26 51	285.581	016 27 04	285.576	-13	0.005
4 – 5	108 10 20	414.975	108 10 34	414.968	-14	0.007
5 – 6	202 22 36	224.356	202 22 58	224.345	-22	0.11
6 – 1	189 14 09	150.024	189 14 09	150.004	0	-0.02

Table 3: Traverse misclosed using the high end GPS dataset

Stn	Bearing	Distance	Northing (m)	Easting (m)
1 – 2	298 28 06	130.613	+62.260	-114.824
2 – 3	300 40 46	291.631	+148.800	-250.823
3 – 4	016 26 04	285.576	+273.885	+80.858
4 – 5	108 10 34	414.968	-129.445	+394.277
5 – 6	202 22 58	224.345	-207.443	-85.411
6 – 1	189 14 09	150.004	-148.059	-24.079
		∑1497.137	Δ 0.002	Nil

With respect to Table 2 and 3, it is shown that the discrepancy for the derived GPS dataset is within the 2nd class cadastral survey requirement (Che, 1996).

Table 4 shows the difference in the derived bearing and distance between traverse stations and the low end GPS. Referring to Table 4.0, it is shown that the discrepancies between bearing and distance are less than 30” and 0.015 m respectively.

Table 5 depicts the accuracy for the closed traverse using the low end GPS and the computed misclosed is 1: 748 583.

Table 4: Difference in bearing and distance (low end GPS)

STN	TRAVERSE		TRIMBLE		DIFFERENCE	
From-To	Bearing	Dist(m)	Bearing	Dist(m)	Azimuth(“)	Dist(m)
1 – 2	298 28 26	130.625	298 28 51	130.624	-25	0.001
2 – 3	300 40 36	291.634	300 40 42	291.631	-6	0.003
3 – 4	016 26 51	285.581	016 27 06	285.574	-15	0.007
4 – 5	108 10 20	414.975	108 10 34	414.964	-14	0.011
5 – 6	202 22 36	224.356	202 23 01	224.343	25	0.013
6 – 1	189 14 09	150.024	189 13 59	150.029	10	-0.005

Table 5: Traverse misclosed using the low end GPS dataset

Stn	Bearing	Distance	Northing (m)	Easting (m)
1 – 2	298 28 51	130.624	+62.290	-114.815
2 – 3	300 40 42	291.631	+148.795	-250.816
3 – 4	016 27 06	285.574	+273.882	+80.876
4 – 5	108 10 34	414.964	-129.443	+394.258
5 – 6	202 23 01	224.343	-207.440	-85.431
6 – 1	189 13 59	150.029	-148.085	-24.072
		∑1497.165	Δ 0.002	Nil

Referring to Table 2 to Table 5, it is shown that there is no significant difference in the derived bearing and distances for the surveyed line for the dataset.

Figure 3 and 4 shows the difference in the derived bearing and distance between the high and low end GPS with respect to the terrestrial dataset respectively. Referring to Figure 3.0, it is shown that there is there is a consistency in the *magnitude* for the derived bearing between the two GPS configuration. It is also shown that the difference for the derived bearing is not significant which is less than 25 second.

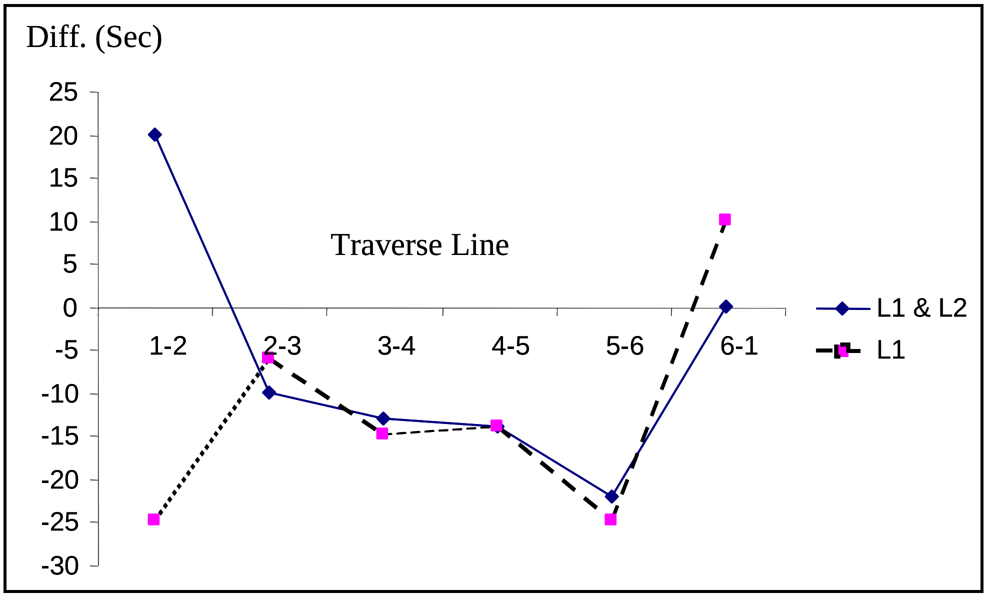

Figure 3: Difference in the derived bearing for the high and low end GPS with respect to the terrestrial dataset.

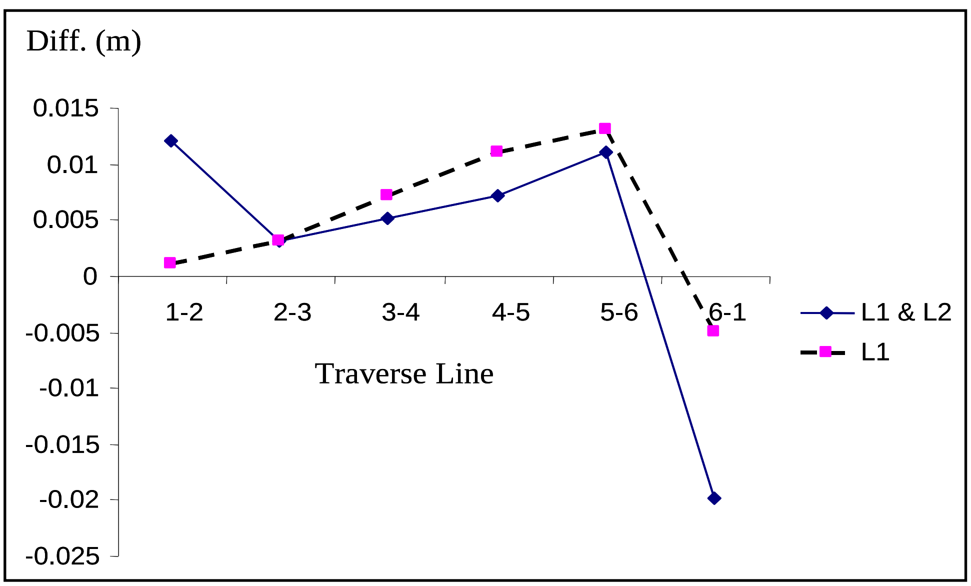

Figure 4: Difference in the derived distance for the high and low end GPS with respect to the terrestrial dataset

Referring to Figure 4, the difference in the derived distance between the two GPS configuration is less than 0.02m. It is also shown that the *magnitude* is almost consistence between the two GPS configuration for the traverse line. It should be noted that, the difference in the derived bearing and distance between the two GPS configuration are not significant, this might due to the short distances between the observed points and therefore contribution of L2 code dataset is almost negligible (Rizos, 1996). Further study on the effect of L2 with respect to various dataset will be carried out to support this claim.

Apart from bearing and distance, in this study the points coordinates are also derived. The main aim of computing these coordinates is to analyze the effect of shift in the Easterly and Northerly direction. Two techniques are adopted in deriving the coordinates using the respective software.

In the first technique, survey Station 1 is held fixed, since the determination of coordinates is based on WGS84, the known Cassini coordinate for Station 1 is first transform to WGS84 through MRSO (Malaysian Rectified Skew Orthomorphic) and MRT (Malayan Revised Triangulation). Having computed all other coordinates in WGS84, the respective coordinates for survey stations are then transform back to the Cassini system (Mohamad, 1994).

Table 6 shows the derived coordinates using the high end GPS with respect to computed coordinates using the terrestrial method. Table 7 depicts the derived coordinates using the low end GPS with respect to the terrestrial coordinates.

Table 6: Coordinates of survey station using the high end GPS (Station 1 held fixed)

STN	TRAVERSE		FIX STN.1		DIFF.	
	(+)N / S (-)	(+)E / W (-)	(+)N / S (-)	(+)E / W (-)	N/S	E/W
1	-11008.277	-22851.176	-11008.278	-22851.173	0.001	-0.003
2	-10946.000	-22966.000	-10945.989	-22965.991	-0.011	-0.009
3	-10797.210	-23216.823	-10797.190	-23216.804	-0.020	-0.019
4	-10523.316	-23135.964	-10523.305	-23135.930	-0.011	-0.034
5	-10652.736	-22741.687	-10652.749	-22741.668	0.013	-0.019
6	-10860.198	-22827.098	-10860.192	-22827.097	-0.006	-0.001

Table 7: Coordinates of survey station using the low end GPS (Station 1 held fixed)

STN	TRAVERSE		FIX STN.1		DIFF	
	(+)N/ S (-)	(+)E / W (-)	(+)N / S (-)	(+)E / W (-)	N/S	E/W
1	-11008.277	-22851.176	-11008.278	-22851.174	0.001	-0.002
2	-10946.000	-22966.000	-10945.986	-22965.990	-0.014	-0.010
3	-10797.210	-23216.823	-10797.191	-23216.806	-0.019	-0.017
4	-10523.316	-23135.964	-10523.309	-23135.930	-0.007	-0.034
5	-10652.736	-22741.687	-10652.752	-22741.672	0.016	-0.015
6	-10860.198	-22827.098	-10860.193	-22827.101	-0.005	0.003

Referring to Table 6 and 7, it is shown that the difference in the N/S and E/W direction for both high and low end GPS is less than ± 0.020 m and ± 0.034 m respectively. In general, all points experience a difference is less than 0.030 m in both directions.

In the second technique, two-survey stations are held fixed for the purpose of adjustment. The survey stations that held fixed are Station 1 and Station 2.

Table 8 shows the derived coordinates using the high end GPS with respect to computed coordinates using the terrestrial method. Table 9 depicts the derived coordinates using the low end GPS with respect to the terrestrial coordinates with respect to the second technique.

Table 8: Coordinates of survey station using the high end GPS (Stn. 1 & 2 held fixed)

STN	TRAVERSE		FIX STN.1 & STN.2		DIFF.	
	(+)N / S (-)	(+)E / W (-)	(+)N / S (-)	(+)E / W (-)	N/S	E/W
1	-11008.277	-22851.176	-11008.279	-22851.173	0.002	-0.003
2	-10946.000	-22966.000	-10946.001	-22965.995	0.001	-0.005
3	-10797.210	-23216.823	-10797.199	-23216.807	-0.011	-0.016
4	-10523.316	-23135.964	-10523.312	-23135.930	-0.004	-0.034
5	-10652.736	-22741.687	-10652.754	-22741.668	0.018	-0.019
6	-10860.198	-22827.098	-10860.193	-22827.096	-0.005	-0.002

Table 9: Coordinates of survey station using the low end GPS (Stn. 1 & 2 held fixed)

STN	TRAVERSE		FIX STN. 1 & STN. 2		DIFF.	
	(+)N / S (-)	(+)E /W(-)	(+)N / S (-)	(+)E / W(-)	N/S	E/W
1	-11008.277	-22851.176	-11008.278	-22851.174	0.001	-0.002
2	-10946.000	-22966.000	-10946.000	-22965.997	0.000	-0.003
3	-10797.210	-23216.823	-10797.235	-23216.832	0.025	0.009
4	-10523.316	-23135.964	-10523.343	-23135.988	0.027	0.024
5	-10652.736	-22741.687	-10652.740	-22741.714	0.004	0.027
6	-10860.198	-22827.098	-10860.190	-22827.119	-0.008	0.021

Referring to Table 8 and 9, it is also shown that the difference in the N/S and E/W direction for both high and low end GPS is less than ± 0.027 m and ± 0.034 m respectively. In general, all points experience a difference less than 0.030 m in both directions and can be accepted for 2^{nd} class cadastral survey.

The main aim of fixing Station 1 & 2 in the adjustment procedure is to ‘imitate’ the general procedure in executing cadastral survey. Based on the study, in general, it is found that fixing a station or two known station does not contribute any significant in the derived coordinates using the high or low end GPS.

However, the issues related to Cadastral survey arise if one used Certified Plan (CP) or GPS Network coordinates (JUPEM, 1994) as datum during its data processing. Table 10.0, clearly shows the difference in derived coordinates of the traverse points prosess from CP’s and GPS Network. Based on this finding, a clear concept of the coordinates system used in Malaysia needs to be studied and understand.

Table 10: Differences in coordinates from CP's and GPS Network used as datum.

	CP TRAVERSE		GPS TRAVERSE		DIFF. (m)	
STN	(+)N / S (-)	(+)E /W(-)	(+)N / S (-)	(+)E / W(-)	N/S	E/W
1	-11008.277	-22851.176	-11010.041	-22849.856	1.764	-1.320
2	-10946.000	-22966.000	-10947.747	-22964.674	1.747	-1.326
3	-10797.210	-23216.823	-10798.951	-23215.489	1.741	-1.334
4	-10523.316	-23135.964	-10525.070	-23134.613	1.754	-1.333
5	-10652.736	-22741.687	-10654.513	-22740.354	1.777	-1.351
6	-10860.198	-22827.098	-10861.955	-22825.784	1.757	-1.314

The experience gained in this study include the following:

1. There is no significant difference in the derived bearing and distance between the high and low end GPS for the surveyed area,
2. The *accuracy* of the derived value (bearing & distance) is applicable for cadastral survey (2^{nd} class) and engineering work,
3. Further study in the effect of longer distances, repeatability of survey, and more number of survey stations (Ses, et.al, 1999) should be explored using the high end and low end GPS,
4. The method of Least square adjustment (Wolf, 1997) and Transformation of coordinates for the derived bearing and distance could be an interesting area to be investigated,
5. Utilization of GPS in monitoring of the environment and structures needs to be Explored (Hartinger, 2000),
6. Further study on the utilization of single GPS receiver and MASS (MASS, 2002) data for mapping survey,
7. The study of height determination using GPS needs also be explored (Sazali, 1996 and Kamaluddin, 2001).

Conclusion

The method of deriving bearing and distance using GPS is outlined. The accuracy of the derived bearing and distance is assessed quantitatively by comparing to the conventional terrestrial survey dataset. The capability of the high and low end GPS towards point positioning is discussed.

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