AN EVALUATION OF SOME STOP-AND-GO KINEMATIC GPS SURVEY OPTIONS

Glenn Wylde, BSurv(Hons) is a recent graduate of the School of Surveying and Land Information.

Will Featherstone, BSc(Hons), D.Phil, MISAust, MAIC is Senior Lecturer in Geodesy at the School of Surveying and Land Information.

GLENN P. WYLDE & WILL E. FEATHERSTONE

School of Surveying and Land Information

Curtin University of Technology

Perth, Western Australia

ABSTRACT

Several stop-and-go kinematic GPS methods were used to coordinate a series of eight points over a one square kilometre area. Almost all coordinates agreed to within a few centimetres of one another, and with a network established using static GPS. However, several outliers (greater than 0.1m in at least one dimension) were identified on subsequent analysis, which could not be quantified at the time of the surveys. This highlights the need for quality assurance in GPS surveys in order to identify gross errors. Recommendations are made on survey procedures which may aid such identification.

INTRODUCTION

The application of the Navstar Global Positioning System (GPS) to surveying and geodesy has evolved rapidly since the early 1980s. When used in the relative carrier-phase mode (ie. interferometrically), GPS offers centimetre-level relative positioning (Hofmann-Wellenhof *et al.*, 1993; Seeber, 1993).

The main obstacle to this method of precise positioning is the determination of the integer ambiguities, which are the unmeasured number of carrier wavelengths between the GPS antenna and satellites. The algorithms used to solve these integer ambiguities have evolved to the extent that GPS stations now need only be occupied for a matter of seconds.

For example, the stop-and-go method (Minkel, 1989), also called semi-kinematic (Cannon and Schwarz, 1989), reduces each in a series of station occupations typically to a minute or so, and offers centimetre-level relative precision. This mode of GPS surveying is a fast and efficient approach to the coordination of a series of points.

For relative kinematic surveys, the integer ambiguities have to be initialised at the start of the survey from what is essentially a known baseline (Hofmann-Wellenhof *et al.*, 1993, p. 141). Therefore, a fundamental requirement of kinematic-based methods is that continuous lock be maintained on at least four GPS signals in order to preserve these ambiguities. If lock is lost to one of this minimum number of satellites, a new integer ambiguity must be determined. Conventionally, this is achieved by returning to a known baseline, which can be inconvenient and therefore best avoided.

However, the advent of fast ambiguity resolution techniques, such as on-the-fly (Abidin, 1994; Hwang, 1991) or FARA (Frei and Beutler, 1990), now enable the ambiguities to be solved without the need for a known baseline (Remondi, 1991). For detailed discussions on the solution of the integer ambiguities, refer to Hofmann-Wellenhof *et al.* (1993, p.190-201) and Seeber (1993, p.264-270) and the references cited therein. The continual refinement of these algorithms will significantly reduce the practical limitations of kinematic-based GPS positioning.

The investigation on which this account is based (Wylde, 1994) evaluates seven relative carrier-phase stop-and-go GPS options, using different software and hardware permutations. Comparisons are made between the resulting coordinate sets themselves, and with a network of eight control points established using relative carrier-phase static and rapid-static GPS. It will be demonstrated that the quality assurance of GPS baselines is an essential consideration, especially if centimetre accuracy GPS positioning is to be achieved on a routine basis.

EXPERIMENTAL DESIGN

The relative carrier-phase stop-and-go kinematic GPS methods under investigation were chosen because of their common use, and the availability of Trimble™ 4000SSE Geodetic System Surveyor GPS receivers and software (Talbot, 1992). The methods evaluated comprise different permutations of hardware and processing software:

- Stop-and-go post-processed using two software packages.
- Two stop-and-go occupation periods post-processed with on-the-fly software.
- RTK (real-time kinematic) with subsequent post-processing of the same data using two software packages.

Observations were made on different days and at different times in order to achieve a variation of satellite geometry. It is considered that such an approach gives a representative sample of the conditions that would be experienced by any GPS user.

The survey area was chosen in order to typify the practical limitations which may be experienced in a suburban environment, most notably overhead obstructions to the GPS signals. The permanent control stations were deliberately located at points where loss of satellite lock was inevitable when moving the antenna between them. However, each station was located with a clear view of the sky above 15 degrees' elevation. These criteria were satisfied on Curtin University's Bentley Campus.

ESTABLISHING THE CONTROL STATIONS

Eight stations were coordinated over a one square kilometre area and used as control for this experiment. Each station was monumented with a brass peg set in concrete and then occupied using the stop-and-go methods under evaluation.

FIGURE 1

The network of GPS-observed baselines used to establish the eight control stations

Figure 1 shows the network of rapid-static GPS baselines (and static GPS in some instances) used to establish accurate coordinates of the eight control stations. Three standard survey marks of the Western Australian Department of Land Administration's geodetic network enabled three-dimensional WGS84 coordinates (DMA, 1987) to be derived for all stations.

All baselines observed in order to establish the control stations were less than 2.5km in length and post-processed using Trimble's GPSurvey™ (version 1.10), then network-adjusted using GeoLab™ (version 2.4d).

In order to pass a χ^2 test, it was necessary to weight the GPSurvey-derived variance-covariance matrices by 240. This is most probably due to an over-optimistic internal estimate of the precision of the GPS baseline vectors when combined with the terrestrial network (Ananga *et al.* 1994). The resulting network-adjusted coordinates did not differ from those produced when no weighting was applied. No outliers, or gross errors, were present in this network. The final three-dimensional WGS84 coordinate set is estimated to be accurate to within 19mm (1σ) in the horizontal and 24mm (1σ) in the vertical.

The GPS-derived WGS84 geodetic coordinates are converted to Universal Transverse Mercator (UTM) easting and northing for ease of presentation. WGS84 ellipsoidal heights are retained as only GPS methods are being compared, and the inclusion of geoid-ellipsoid separations does not affect the conclusions reached. If Australian Geodetic Datum (AGD) or Australian Map Grid (AMG) and Australian Height Datum (AHD) coordinates are required, the procedures of Steed (1990) could be followed.

TABLE 1

Final network-adjusted WGS84 coordinates of the eight control stations, expressed as zone 50 UTM easting, nothing and ellipsoidal height (units in metres).

ID	UTM N	UTM E	ellip. ht.
A	6458478.176	395480.967	2.047
1	6459234.188	395231.058	-22.820
2	6458869.259	394981.507	-24.224
3	6458669.679	395041.875	-22.200
4	6458490.110	395132.015	-22.832
5	6458325.222	395179.293	-23.245
6	6457654.429	395316.967	-23.093
7	6458044.962	395032.633	-23.643
8	6457946.888	394923.019	-24.718

FIELD PROCEDURES

All stop-and-go surveys were undertaken during August 1994 on days when the PDOP was less than five. GPS data were recorded from all visible satellites with a greater than 15 degree elevation. The manufacturer's manuals were followed for the operation of hardware, and all software was used with default parameter settings. Therefore, no *a priori* assumptions of the performance of the equipment were made which could bias the final results.

All stop-and-go surveys were conducted radially from the common reference point *A* (Figure 1). The observed baselines were less than 850m in length. The subsequent analyses are, therefore, a comparison of *radial* stop-and-go GPS observations with *networked* points observed using static GPS. This is an important factor, upon which some of our conclusions and recommendations are based.

Obviously, minor operational procedures varied between the different user manuals, which will not be duplicated here. However, the general survey scheme was as follows:

- Position the reference antenna over point A (Figure 1) and record kinematic GPS data continuously throughout the survey.
- Position the roving antenna over the control station for between two and five minutes and record kinematic GPS data.
- Move the roving antenna to the next control station, whilst continuously collecting GPS data,

and attempting not to pass under obstructions which could possibly cause loss of satellite lock.
- Repeat the previous two procedures until all stations have been occupied.

Essentially, two files of kinematic GPS data are recorded for each complete survey; one for the reference station and one for the remote stations.

STOP-AND-GO POST-PROCESSED

The GPS data were collected at five-second epochs using the Trimble 4000SSE receivers. When loss of satellite lock was encountered on roving between stations (usually indicated by the receiver), the ambiguities were reinitialised on one of the known control stations. It may be argued that this would introduce improved comparisons at these stations. However, on inspecting the results, no such correlation is identifiable. In Tables 2 and 3, loss of lock occurred between stations 1 and 2, 5 and 6, 6 and 7, and 7 and 8.

The recorded data were then down-loaded and post-processed using (i) GPSurvey™ version 1.10, and (ii) Trimvec Plus™ revision E. The residuals (test minus control coordinates in each dimension) are shown in Tables 2 and 3 respectively.

TABLE 2

Coordinate differences at the eight control stations for stop-and-go GPS, post-processed with GPSurvey (units in metres).

ID	resid. N	resid. E	resid. ht.
1	-0.013	0.005	0.013
2	0.037	-0.003	-0.019
3	0.568	0.564	2.388
4	0.056	-0.363	0.946
5	-0.020	0.001	-0.029
6	0.045	0.029	-0.003
7	-0.004	0.030	0.015
8	-0.003	-0.005	-0.031

TABLE 3

Coordinate differences at the eight control stations for stop-and-go GPS, post-processed with Trimvec Plus (units in metres).

ID	resid. N	resid. E	resid. ht.
1	-0.001	0.003	0.006
2	0.027	-0.005	0.008
3	0.156	0.076	0.126
4	-0.023	-0.003	-0.047
5	-0.012	-0.003	-0.019
6	0.032	0.001	-0.012
7	0.009	0.014	0.013
8	0.001	-0.017	0.017

The residual coordinates for points 3 and 4 in Table 3 and point 3 in Table 2 are greater than 0.1m in at least one dimension. These are deemed to have failed because the stop-and-go GPS is expected to yield centimetre-level relative positioning, especially over 850m baselines. This highlights the need for quality assurance of these baselines, otherwise the GPS user will be unsure as to the location or magnitude of these gross errors. Furthermore, the results in Tables 2 and 3 were derived from the same observational data via two different post-processing software packages. One would expect identical results, but there remain centimetre-level differences between these positions.

STOP-AND-GO ON-THE-FLY

By definition, on-the-fly refers to the solution of the integer ambiguities whilst the GPS antenna is in motion, using a single epoch of data (Hofmann-Wellenhof *et al.*, 1993, p. 175). Therefore, the on-the-fly approach gives positions on an epoch-by-epoch basis. There is now no requirement to maintain satellite lock when moving between stations or to reinitialise from a known baseline.

The stop-and-go procedure used here simply takes the average of the on-the-fly post-processed epoch-by-epoch coordinates at each station. Unfortunately, statistical analyses, with which to study the behaviour of the solutions at each station, were unavailable.

The kinematic GPS data were collected continuously at a one-second epoch using the

Trimble 4000SSE receivers. These data were down-loaded then post-processed using on-the-fly software developed in the United States (Frodge *et al.*, 1994).

In this experiment, the one-second solutions were averaged over three and five minutes for each station to yield two sets of results. The residuals for the three-minute and five-minute station occupations are shown in Tables 4 and 5 respectively.

TABLE 4

Coordinate differences at the eight control stations for three-minute occupations, post-processed with on-the-fly software (units in metres).

ID	resid. N	resid. E	resid. ht.
1	0.000	0.004	0.020
2	3.510	-1.503	-2.736
3	-0.001	0.014	0.020
4	-0.006	0.001	0.048
5	0.008	0.012	0.015
6	0.024	0.004	0.017
7	-0.008	0.003	0.007
8	-0.006	-0.004	-0.008

TABLE 5

Coordinate differences at the eight control stations for five-minute occupations, post-processed with on-the-fly software (units in metres).

ID	resid. N	resid. E	resid. ht.
1	0.009	0.007	0.020
2	-0.007	-0.003	-0.019
3	-0.013	0.006	0.020
4	-0.007	0.014	0.038
5	0.055	-0.001	0.015
6	0.039	-0.002	0.007
7	-0.746	0.164	-0.563
8	-0.009	-0.015	0.022

Again, outliers are identified at points 2 and 7 in Tables 4 and 5 respectively, when compared with one another and with the control station coordinates. Furthermore, these failed points are different from the first occupation which does not indicate the source of error to be in the control network. As with the previous test, the user would be unaware of these gross errors during such a radial stop-and-go survey.

REAL-TIME KINEMATIC (RTK)

To facilitate real-time kinematic (RTK) positioning, carrier-phase data must be transmitted from the reference receiver to the roving receiver before processing and coordinate display. As some time is required for these, real-time GPS positioning is not instantaneous.

The RTK surveys were performed with the Trimble 4000SSE receivers in conjunction with a Trimble TDC1™ controller and Trimtalk™ telemetry system. The one-second epoch data were simultaneously recorded for subsequent post-processing using GPSurvey and Trimvec Plus. When loss of satellite lock was encountered, the antenna was held stationary for approximately one minute in order to allow ambiguity resolution. As the antenna is not in motion, this is not on-the-fly ambiguity resolution, but does avoid the requirement for a known baseline. However, the known stations were also occupied so as to enable the post-processing software to resolve the ambiguities.

Some difficulties were encountered with the telemetry system in that radio lock was lost, usually due to obstructions between the reference and roving receivers. This resulted in a change of the integer ambiguities. Therefore, radio communication had to be re-established, followed by ambiguity reinitialisation.

The RTK residuals are given in Table 6, GPSurvey in Table 7, and Trimvec Plus in Table 8.

TABLE 6

Coordinate differences at the eight control stations for RTK occupations (units in metres).

ID	resid. N	resid. E	resid. ht.
1	-0.022	0.004	-0.022
2	0.010	0.011	0.015
3	0.001	0.008	0.001
4	-0.021	-0.004	0.021
5	-0.010	-0.021	-0.013
6	0.036	-0.003	0.004
7	-0.002	0.031	-0.029
8	-0.010	0.006	-0.024

The RTK system prompted for horizontal and vertical position tolerances to be specified at the start of the survey. These were set at 15mm and 20mm respectively. On inspecting the residuals in Table 6, these criteria were met in only two of the eight instances. However, this comparison does not take into account the uncertainties in the control coordinates.

TABLE 7

Coordinate differences at the eight control stations for RTK occupations, post-processed with GPSurvey (units in metres).

ID	resid. N	resid. E	resid. ht.
1	-0.039	0.013	-0.017
2	0.026	0.033	0.001
3	0.002	0.018	0.031
4	-0.018	0.000	0.062
5	-0.005	-0.012	-0.005
6	0.031	-0.003	0.046
7	0.000	0.031	-0.005
8	-0.008	-0.004	0.001

TABLE 8

Coordinate differences at the eight control stations for RTK occupations, post-processed with Trimvec Plus (units in metres).

ID	resid. N	resid. E	resid. ht.
1	-0.025	0.007	-0.003
2	0.010	0.013	0.008
3	0.003	0.007	0.020
4	-0.017	0.001	0.045
5	-0.014	-0.019	-0.014
6	0.034	-0.005	0.033
7	0.000	0.030	-0.007
8	-0.005	0.003	0.006

The residuals in Tables 7 and 8 use the same GPS data as those in Table 6 and the coordinates of each point agree to within 20mm. The remaining discrepancies are due purely to the different processing algorithms employed in each software package.

Also, none of the results from this survey were deemed to have failed. It is expected that this is because the RTK system collects sufficient GPS data in order to determine a satisfactory position, before prompting the user to move to the next point.

DISCUSSION

The mean and root mean square (rms) residuals of the seven evaluations in Tables 2 to 8 are summarised in Table 9. Figures 2 and 3 show the same data as bar charts. These tables and figures exclude the points deemed to have failed (ie. values whose residual is greater than 0.1m in at least one dimension). The number of failed points is implied in column five of Table 9.

For ease of presentation, the following abbreviations are used: PPG - stop-and-go post-processed with GPSurvey, PPT - stop-and-go post-processed with Trimvec Plus, OTF5 and OTF3 - five and three minute occupation post-processed using on-the-fly software, RTK - real-time kinematic, RPPG - real-time kinematic data post-processed with GPSurvey, RPPT - real-time kinematic data post-processed with Trimvec Plus.

TABLE 9

Mean and root mean square coordinate differences (excluding outliers) at the eight control stations for all seven stop-and-go options (units in metres).

option	mean N (rms N)	mean E (rms E)	mean ht. (rms ht.)	no pts
PPG	0.015 (0.017)	0.012 (0.013)	0.018 (0.010)	6
PPT	0.015 (0.015)	0.007 (0.006)	0.017 (0.014)	7
OTF5	0.020 (0.019)	0.007 (0.006)	0.020 (0.009)	7
OTF3	0.007 (0.008)	0.006 (0.005)	0.019 (0.014)	7
RTK	0.014 (0.012)	0.011 (0.010)	0.016 (0.010)	8
RPPG	0.016 (0.015)	0.014 (0.012)	0.021 (0.023)	8
RPPT	0.013 (0.012)	0.014 (0.010)	0.017 (0.015)	8

In Tables 2 to 8, the stop-and-go coordinates, which did not fail, agree with one another, and with the control stations, to within 55mm in the horizontal and 48mm in the vertical. The mean and rms differences for do not exceed 23mm horizontal and 21mm vertical. However, these statistics exclude the failed points or outliers, which constitute a considerably more significant result.

The outliers range from 0.174m to 3.818m in the horizontal, and from 0.563m to 2.736m in the vertical. These are randomly scattered between control stations, again indicating that gross errors are not present in the control station coordinates. The most alarming fact is that, at the time of processing, no firm indication was given as to the quality of the resulting coordinates. There is one exception in that the GPSurvey software, which gives ratio and variance indicators (Trimble, 1993; Hofmann-Wellenhof *et al.*, 1993, p.148).

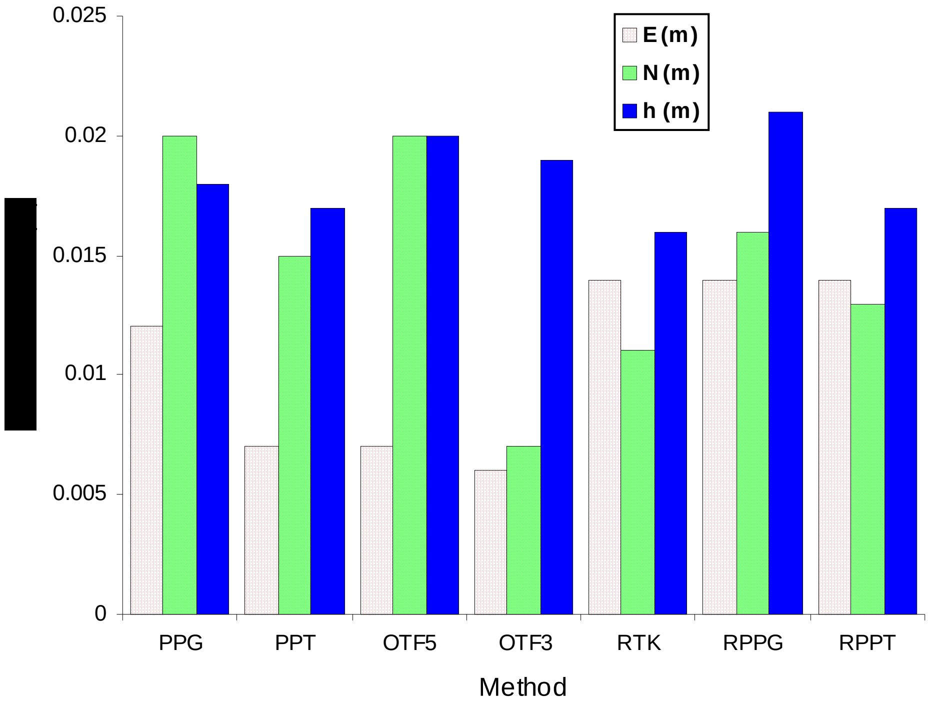

FIGURE 2

Mean residuals for each of the evaluated options

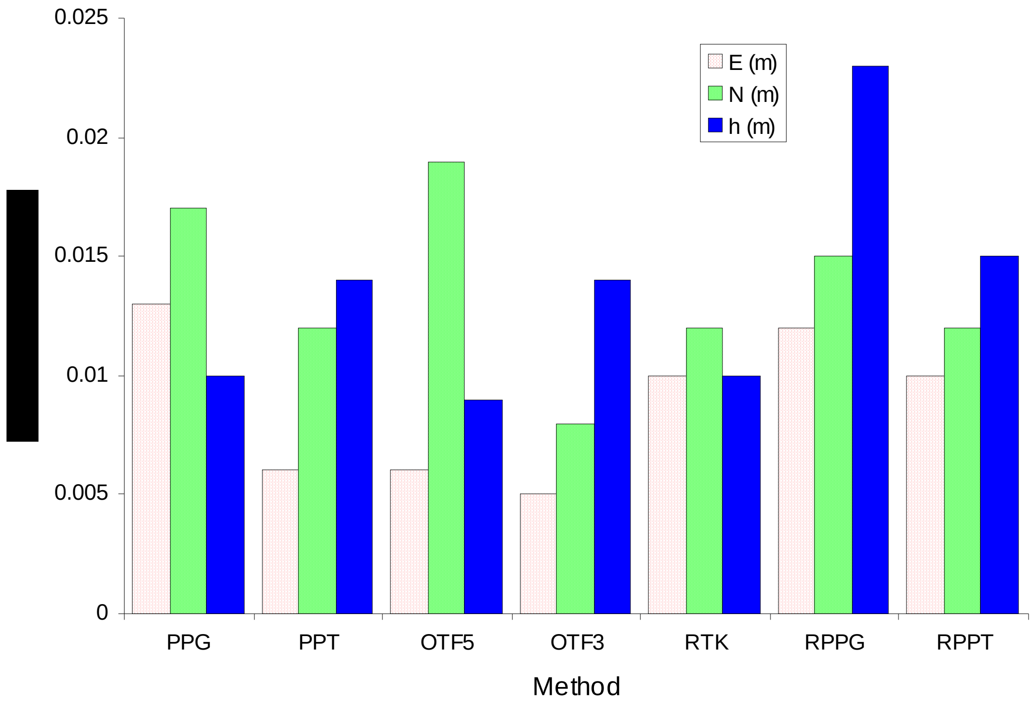

FIGURE 3

Root mean square residuals for each of the evaluated options

The five outliers were only identified upon subsequent comparisons between each stop-and-go method, and with the network of control stations. Had we simply undertaken a single stop-and-go survey, we would have not been aware that some of our positions were in error by over three metres. This highlights the need for quality assurance during stop-and-go kinematic surveys in order to identify such gross errors.

An equally important observation is that the same GPS data can give results that are in gross error, depending upon the processing software used. For example, some positions did not fail with one package, whereas they did with another, and *vice versa*. This illustrates that all GPS software can not necessarily be assumed to deliver results of equal accuracy. This applies to *all* GPS software, not only the limited number of options evaluated here. One suggestion to address this deficiency is that a 'benchmark' GPS raw data set be used to test various GPS algorithms. This would lead to standardised procedures, which deliver specified levels of positional accuracy.

Some of the centimetre-level differences between options may be attributed to random centring errors of the antenna over each control point. It is acknowledged that by collecting all data simultaneously would remove such errors, but this posed a logistical problem. However, it is reiterated that these surveys were carried out, as any GPS practitioner would, by following documented instructions. Moreover, any antenna centring errors do not account for the observed metre-level gross errors.

It may also be argued that eight points do not constitute a statistically significant sample size with which to reach a universal conclusion. However, one failed result is sufficient to raise doubt as to the validity of any method.

Another important observation is the residuals for station 6. Each stop-and-go method is approximately offset by 30mm due East of this station. The good repeatability of the stop-and-go methods (20mm horizontal agreement with one another) indicates that the quality of this control station is suspect. On inspection of Figure 1, station 6 has a poor network geometry. This confirms that an optimum network geometry should always be used for GPS surveys.

RECOMMENDATIONS

This relatively small experiment raises two important issues which must be taken into consideration by the stop-and-go kinematic GPS surveyor, but are equally applicable to all GPS surveyors.

The most important consideration is that there exists a need for quality assurance in radial GPS surveys. Due to the nature of kinematic surveys, they are usually conducted radially and it is widely acknowledged that such surveys may contain undetectable gross errors.

This problem is also identified by Hofmann-Wellenhof *et al.* (1993, p.148): "The main check for kinematic vectors is to compute positions and check that similar values are obtained on separate visits to the same point. Also, it is good survey practice to visit points whose coordinates are known during the survey as a further check on the method."

One approach would be to use two reference receivers simultaneously with one roving receiver. This would facilitate a three-station network per kinematic point. However, any GPS integrity errors would be common at the roving receiver, and may not necessarily be identified using this approach alone. Alternatively, a network could be established using a series of stop-and-go occupations from several reference stations. This would aid the identification of outliers and a network adjustment may improve the final coordinates. However, these methods would incur additional hardware or survey costs.

The second consideration is that, if GPS networks are to be used, they must be of good geometric strength. This is in order to detect gross errors and eliminate those points which are not sufficiently accurate.

GPS surveying is essentially a three-stage process comprising observation and processing; analysis of data reliability; and, re-observation and reprocessing, if necessary. Terrestrial surveying has always relied on redundancy, careful observation and data reduction, and GPS surveying should be no exception, especially when striving for centimetre-level positioning.

DISCLAIMER

The software and hardware used in this experiment are primarily from the same manufacturer. These tests are in no way a reflection upon this manufacturer, rather they reflect GPS procedures in general. This discussion is aimed at highlighting the need for quality control in GPS surveys. The manufacturer's name was only included so as to give a full account of the tests presented.

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REFERENCES

Abidin, H.Z. (1994) On-the-fly ambiguity resolution, GPS World, 5(4), 40-50.

Ananga, N., R. Coleman and P. Morgan (1994) Variance-covariance estimation for GPS networks, Bulletin Geodesique, 68, 77-87.

Cannon, M.E. and K-P. Schwarz (1989) GPS semi-kinematic positioning along a well-controlled traverse, Proceedings of the Fifth International Geodetic Symposium on Satellite Positioning, Las Cruces, New Mexico, 621-630.

DMA (1987) Department of Defense World Geodetic System 1984: its definition and relationships with local geodetic systems. Report 8350.2, Defense Mapping Agency, Washington.

Frei, E. and Beutler, G. (1991) Rapid static positioning based on the fast ambiguity resolution approach: the alternative to kinematic positioning, Proceedings of GPS '90, Ottowa, 1196-1216.

Frodge, S.L., B. Shannon, B. Remondi, D. Lapucha and R.A. Barker (1994) Results of real-time testing of GPS carrier phase ambiguity resolution on-the-fly, proceedings of FIG commission 5, Melbourne.

GeoLab™ (1993) version 2.4d, Bitwise Ideas Inc., 6-1050 Baxter Road, Ottawa, Ontario, K2C 3P1, Canada.

GPSurvey^{TM} (1993) version 1.10, Trimble Navigation Ltd., PO Box 3642, Sunnyvale, California 94088-3642, USA.

Hofmann-Wellenhof, B., H. Lichtenegger and J. Collins (1993) GPS theory and practice (second edition), Springer-Verlag, Wien, New York.

Hwang, P.Y.C. (1991) Kinematic GPS for differential positioning: resolving integer ambiguities on-the-fly, Navigation, 38, 1-15.

Minkel, D.H. (1989) Demonstration and discussion of the pseudo-kinematic method. Proceedings of the Fifth International Geodetic Symposium on Satellite Positioning, Las Cruces, New Mexico, 577-588.

Remondi, B. (1991) Kinematic GPS results without static initialisation. NOAA technical memorandum NOS NGS-52, Rockville, Maryland.

Seeber, G. (1993) Satellite Geodesy: foundations, methods and applications, Walter de Gruyter, Berlin.

Steed, J. (1990) A practical approach to transformation between commonly used reference systems, The Australian Surveyor, 35, 248-264 and 384.

Talbot, N.C. (1992) Recent advances in GPS hardware and software, National Conference in GPS surveying, The University of New South Wales.

Trimvec Plus^{TM} (1993) revision E, Trimble Navigation Ltd., PO Box 3642, Sunnyvale, California 94088-3642, USA.

Wylde, G.P. (1994) An evaluation of stop-and-go GPS survey techniques, Honours dissertation, Curtin University of Technology.