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AN ATTEMPT OF USING ONE FREQUENCY GPS RECEIVERS TO DETERMINATION OF GEODETIC NETWORKS

Створення геодезичних мереж GPS-технологією виконують, як правило, з використанням двочастотних приймачів. Точність визначення пункту власне і регламентує цей вибір. З іншої сторони прогрес в області GPS технологій приводить до ще вищої точності визначення координат. У цій статті дається аналіз результатів проведення спостережень з використанням одночастотних приймачів. Зроблена оцінка точності згідно польських технічних рекомендацій.

Survey of geodetic networks by GPS technique is usually being performed with the use of two frequency receivers. The accuracy of a point determination is of course the main reason of the selection of receiver type. On the other hand however, the dynamic technical development in the area of GPS receivers as well as the application of the improved methods of postprocessing (i.a. thanks to considering the new achievements in study on ionospheric and tropospheric models) lead to higher and higher accuracy of the final product that are point coordinates. In this paper there is presented the analysis of survey results obtained with the use of one frequency receivers. Accuracy assessment has been done with reference to current Polish technical rules (guide lines).

1. Introduction. In geodetic applications the GPS technology is mainly based on two frequency receivers (i.e. signal of carrier wave is being measured with frequency of L1 and L2 [Lamparski 2001]). The receivers determining position based only on frequency L1 are first of all used for navigation purposes [e.g. Martin-Mur and Dow 1997]. Currently, however, the attempts of apply of one frequency receivers to geodetic purposes are made often and often [Figurski et al. 2000]. Such solution is acceptable i.a. thanks to quick technical growth in the area of surveying equipment that makes possible greater control of the measuring process. Development of research on the factors disturbing propagation of GPS signal (e.g. improved ionospheric model considering disturbance periods) is also great of importance. This gives us an opportunity of raising the accuracy of results on the stage of postprocessing. The argument for selection receivers L1 is their price (compared to receivers L1-L2).

The thesis of this work is the possibility of achievement sufficient accuracy of a point coordinates in geodetic networks when using currently available receivers measuring frequency L1 (without P-code) – cf. e.g. [Lamparski 2001]. The criteria to be fulfilled when determining the geodetic networks are given by technical regulations [GUGiK 2000, 2001].

2. Description of surveying equipment. The test network was measured using one frequency GPS receiver Thales ProMark3. This model finds application in some geodetic as well as in GIS measurements. The data received from satellites are following:

- carrier frequency L1,
- C/A code (*Coarse/Acquisition*) – freely available navigation code with short repeat period (1 milisecond),
- phase of carrier wave,
- WAAS – (*Wide Area Augmentation System*) – American system of differential corrections transmitted by geostationary satellites,
- EGNOS – (*European Geostationary Navigation Overlay Service*) – transmission system of differential corrections by two satellites INMARSAT III and one satellite ARTEMIS.

The receiver ProMark3 can simultaneously track 14 satellites on 14 parallel canals (if they are available). The frequency of refreshing the data from satellites equal 1 Hz. Differential corrections are received in RTCM SC-104 v2.1 format.

All the observations received during geodetic measurements can be numerically processed by means of GNSS SOLUTIONS program. It makes possible planning the surveys, calculation of observations, processing observation data from the receivers of other producers, using the data from reference networks of ASG-PL (*Polish Active Geodetic Network*), adjustment of network by least square method, accuracy assessment, transformation of coordinates to the national sets valid currently in Poland as well as to the local geodetic sets, presentation of data in graphic or table form, generating of the reports from measurements and calculations, and export of results in various formats (also defined by user).

3. Surveys and calculations. In order to determine of coordinate by GPS technology, two methods: *static* and *Stop&Go* were applied. Four GPS receivers Thales ProMark3 were used in measurement process. More detailed description of survey and calculation is presented in work [Pogan and Pieszka 2007].

GPS survey by static method

Static survey was planned with 4 measuring sessions (tab. 1), in which 10 points were measured (cf. sketch on Fig. 1). The data have been collected by a receiver for at least 15 min. on each of points.

Table 1

Plan of measuring sessions for static method

Session No	Signification of a receiver			
	A	B	C	D
1	2048	AR 30	AR 20	2041
2	2042	2044	5000	2041
3	2046	2044	2043	2045
4	2048	AR 30	2043	2045

The horizon limit was set in 10^{o} and survey interval – in 5s. On each position (station) the surveying register has been performed. Such information as the time of starting and ending the measurement as well as height of GPS antenna (type NAP100) have been written down. The survey data were stored in internal memory of a receiver. After transmission of the data to a computer, they were given into numeric processing by means of GNSS SOLUTIONS program.

All things considered there were 23 vectors measured. The longest vector amounted to 238,693m whereas the shortest – 9,629m. The PDOP coefficient varied in a range of 1,5÷1,9. The global accuracy of vectors measured amounted ±3mm. The corrections of vectors after adjustment were situated in a range of 1mm ÷ 10mm, whereas the corrections for the components X, Y, Z amounted from -5mm to 9mm. On the basis of vectors adjusted the horizontal coordinates of points were calculated in the set “2000” (one of the sets valid in Poland). Next the transformation to the set “1965” (zone 1) were performed. The issues of the coordinates conversion between various sets is not a topic of this paper. More information on this topic can be found in numerous publications [e.g. Kadaj 2000, Lamparski 1998].

In order to calculate the altitudes (H) of network points, the known altitudes of points AR20, AR30 and 5000 were used. On the basis of them, the transformation of ellipsoidal heights to the set of "Kronstadt 85” were performed. As the accuracy assessment of determination of the plane coordinates (X, Y) was the aim of this work, only such coordinates will be took into further consideration.

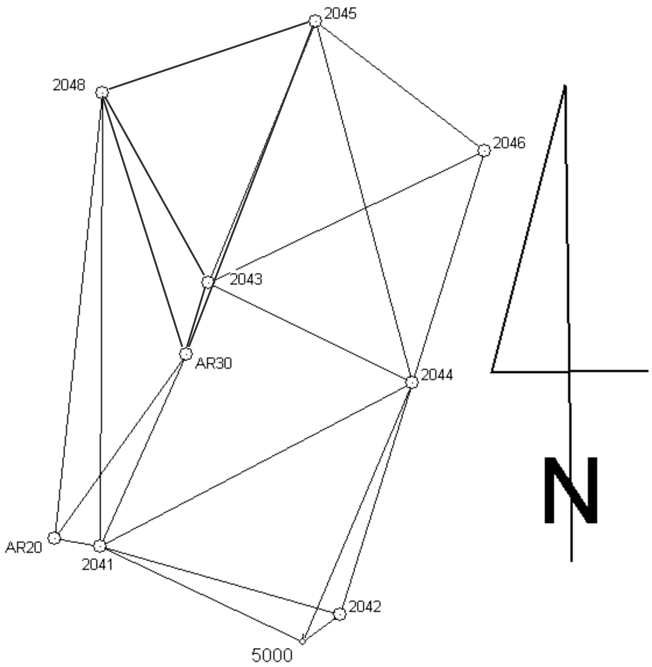

Fig. 1. Sketch of the measurement by static method

GPS survey by Stop&Go method

The survey by this method was done with using of two receivers *Thales ProMark3.* The point No 2048 was adapted as a firm point and therefore the base receiver was put up on it (the antenna height amounted 1,653m). Then, by means of a crossbeam with known length, the initialization lasting 5min. were performed with the aim of phase ambiguity determination. The next step was to move another antenna from the crossbeam to the pole with height measured (2,150m) in the moving receiver. Then the 20 sec. measurements were performed on the consecutive points. There were 11 vectors measured: the shortest one – 0,208 m (initialization on the crossbeam), the longest one – 306,974 m. There appeared a signal disturbance on the point No 2041, therefore the repeated measurement was made.

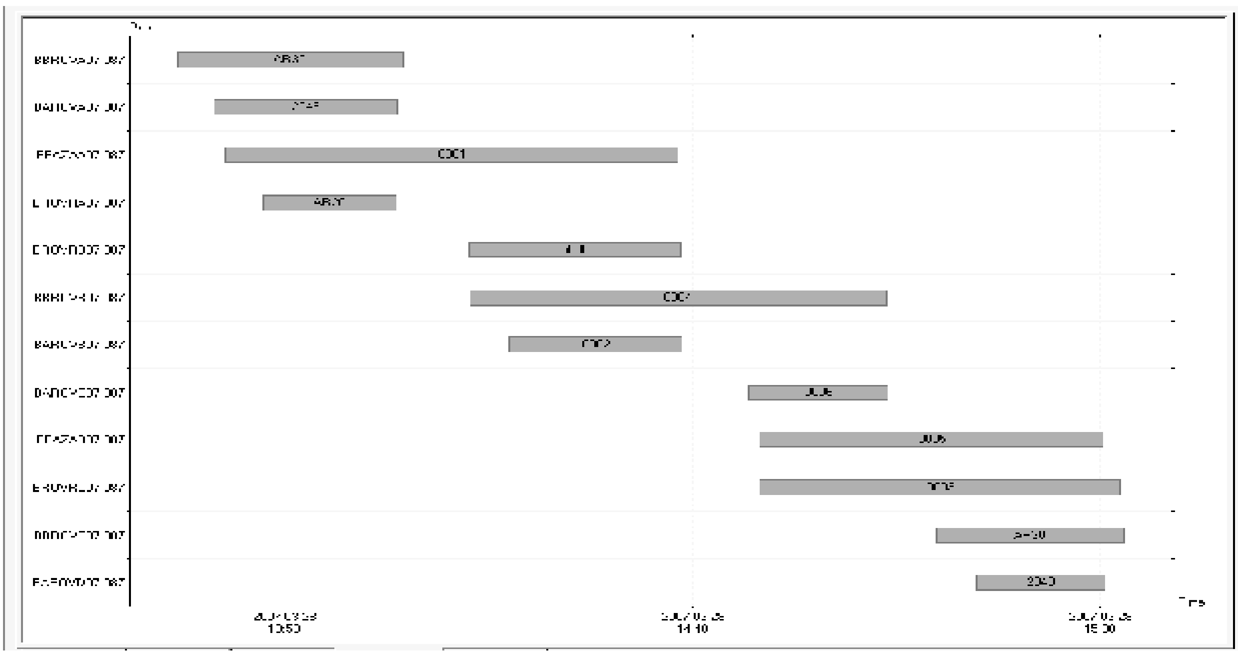

Fig. 2. Distribution of measuring sessions for static method

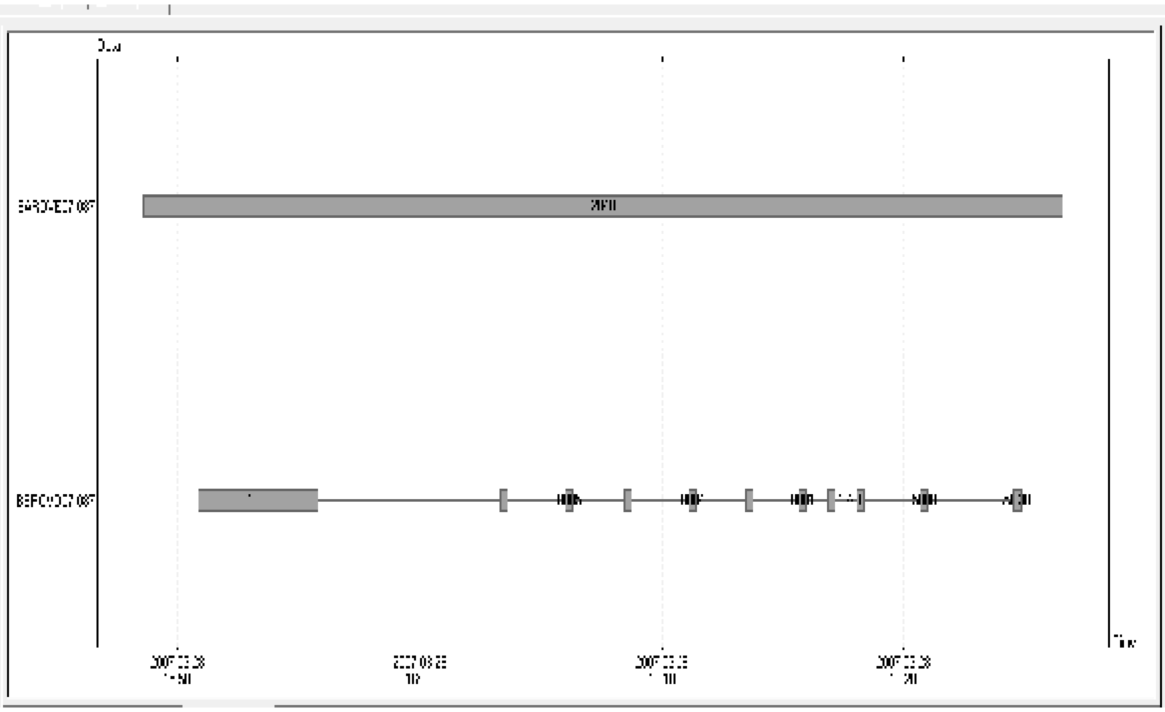

Fig. 3. Distribution of measuring sessions for Stop&Go method

Like in previous method (static one), every calculation were performed with using GNSS SOLUTIONS program. The global accuracy of vectors measured amounted to ±7mm. The PDOP coefficient varied in a range of 1,4÷2,2. The mean error of vector length was situated in limits of 2mm ÷ 14mm, but the components X, Y, Z of vectors were measured with precision of 2mm ÷ 7mm. On the base of the measured and adjusted observations (vectors) the coordinates of points were calculated in “2000” set and next transformed to “1965” set.

4. Results. As an effect of calculating works performed there are the detailed protocols from surveys adjustment process [see: Pogan and Pieszka 2007]. On the base of them there was made an attempt of assessment (comparison) of the accuracy obtained by GPS methods with reference to the classical (terrestrial) survey methods. The results of GPS surveys (static method and Stop&Go) adjustment for a few selected points were compared with the coordinates determined by classical methods (tab. 2-3).

In the light of the Polish technical rules [GUGiK 2002], the accuracy of coordinates determined by both GPS methods are sufficient for survey networks, but even for networks of higher classes (national networks – [GUGiK 2000]). The mean errors m_X, m_Y of coordinates and the positional errors m_P are in order of only a few millimeters. For the point AR30 merely in Stop&Go method these values are a little higher (over 1cm). This was probably caused by the less accurate leveling of the pole with antenna.

Table 2

Comparison of coordinates from the static GPS method and the classical one

Point No	Static GPS method					Classical method		Differences	
	X	Y	m_X	m_Y	m_P	X	Y	$\|V_X\|$	$\|V_Y\|$
5000	5407395,723	4549168,744	0,002	0,002	0,003	5407395,711	4549168,723	0,012	0,021
AR 20	5407444,605	4549026,100	0,003	0,003	0,004	5407444, 621	4549026, 112	0,016	0,012
AR 30	5407541,460	4549093,442	0,002	0,002	0,003	5407541, 481	4549093, 442	0,021	0,000

Significations: X, Y – plane coordinates
m_X, m_Y – mean errors of adjusted coordinates
m_P – positional terror of a point
$|V_X|$, $|V_Y|$ – absolute differences between GPS method and classical one

Table 3

Comparison of coordinates from the Stop&Go GPS method and the classical one

Point No	Stop&Go GPS method					Classical method		Differences	
	X	Y	m_X	m_Y	m_P	X	Y	$\lvert V_X\rvert$	$\lvert V_Y\rvert$
5000	5407395,721	4549168,748	0,002	0,002	0,003	5407395,711	4549168,723	0,010	0,025
AR 20	5407444,600	4549026,111	0,002	0,002	0,003	5407444, 621	4549026, 112	0,021	0,001
AR 30	5407541,464	4549093,445	0,014	0,011	0,018	5407541, 481	4549093, 442	0,017	0,003
Significations: - cf. tab. 1									

In order to assessing of the reliability (dependability) of the obtained results, there are also the coordinates differences (deviations) $|V_X|$, $|V_Y|$ between GPS method and classical one compared in tables 2-3. For the static GPS method these deviations are taking the values of 0÷21mm, but for the Stop&Go method they are only insignificantly higher (3÷25mm). As one can see, also in this case the values of coordinates determined by GPS methods do not arise any reservations.

5. Summary and conclusions

The aim of this work was to find the accuracy of point determination on the plane on the base of GPS surveys when using one frequency receivers Thales ProMark3. The survey of test network by two methods: static and Stop&Go was performed. For numerical development of results (postprocessing, adjustment) the GNSS SOLUTIONS program was used. On the stage of comparing analysis of results, the coordinates of network from classical measurement (by *total station*) were also taken into consideration. As the criteria of accuracy assessment the following parameters were assumed: mean errors of coordinates, positional error and the coordinate deviations with reference to classical method.

Basing on the analysis of the obtained results one can juxtapose the following conclusions:

1) The GPS measurement by receiver L1 ensures the sufficient accuracy (in the area of technical rules requirements) for establishment of a horizontal minor network and the Polish national network of the 3-rd class.

2) The Stop&Go method can be successfully used when determining a horizontal minor control network (m_P < 0,10m).

3) The national networks of the 3-rd class, which are characterized by not only higher precision, but also by the appropriate reliability, should be measured by static method. It is also recommended to apply the longest observation time (over 15 min.).

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