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Aspects Regarding the Implementation of the Support Network in Cojocna Didactic Base UASVM

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Abstract. The paper aims to realize a precision analyze of the support network designed in the Cojocna didactic base and the mathematical models used to establish parameters for the positioning of support network points and precision indices. After the recognition of the area and identifying the points from the state triangulation, to achieve the support network in order to satisfy the necessary precision for stakeout operations and monitoring some objectives from the area, was chosen the use of global positioning technology to restore the support network. Instrumental observations were made with the geodesic class receptors, with single or double frequency. Transforming the coordinates from WGS 84 system into Stereo 1970 system was made using Bursa-Wolf model.

Keywords: support network, triangulation network, global positioning

INTRODUCTION

The realization of the support network in the perimeter of Cojocna didactic base aims to locate and monitor various existing and future goals as well as achieving a geodesic polygon in which students from land measurements and cadastre section will conduct their annual practice.

Besides the practical modalities of the support network realization, the paper proposes an analytic mathematical models used to establish parameters for positioning support network points and indices of accuracy (Noureddine, 2006).

Network support for large-scale plans, or marking targets in area of interest is chosen according to the relief shape and the size of the area. In the case of large areas the support networks are presented as triangulation networks, trilateration or polygonometry (Marzooqi, 2005).

Depending on the terrain relief the support network can take the form of chains of triangles, quadrilaterals with two observed diagonals or central polygons.

Until the appearance of radio-telemeters or electro-optical telemeters, the support networks consisted of triangulation networks, with the starting and closing measure by the invar wire or Ciurileanu wire. Ulterior, increasingly more, were used trilateration network and angular – linear networks (triangulation-trilateration).

In the Cojocna didactic base, after the field recognition were identified four points of order V from the state network where signals were destroyed.

To raise or draw some characteristic points of some targets in the area of interest, it was necessary the restoration and thickening the support network so that will respond to the requirements of precision and visibility.

In cases where it is estimated that the accuracy of determining the position of points in the network state is not enough, there are built local geodetic networks that have inferior positioning errors than the state triangulation.

MATERIALS AND METHODS

To ensure accuracy of work required for lifting and tracing support network design is realized in three steps, measured and independently compensated.

Taking into account the influence of measurement errors in each step we can calculate the total mean square error of positioning the lifting points:

$$M = \sqrt{m_1^2 + m_2^2 + m_3^2} \qquad (1)$$

where:

m_1, m_2, m_3 – medium errors of the measurements for each step.

Planimetric accuracy needed for the support network is projected starting from the permissible tolerance from the normative, which is calculated from the mean square error of positioning points and depending on which measurement methods are selected and used devices.

According to Levciuk, 1970, measurement errors in the superior steps are considered initial data errors or errors of the support network for lower levels.

In order to reduce deformations of the support network from current stage, the upper stage errors should be less than k times the total influence of measurements errors m_{mas} from current stage. Thus we can write:

$$m_s = \frac{m_{mas}}{k} \qquad (2)$$

where:

1/k – coefficient of neglecting the influence of support network errors;

k – coefficient of increasing the measurements precision;

The total medium error M of positioning a point, for each stage of the support network, is calculated with the relation:

$$M^2 = m_s^2 + m_{mas}^2 = m_{mas}^2\left(\frac{1}{k^2} + 1\right) \qquad (3)$$

where :

m_s - initial data errors;

m_{mas} - measurement errors.

From expression (3) is deducted the general relation for calculating the precision coefficient k:

$$k = \frac{m_{mas}}{\sqrt{M^2 - m_{mas}^2}} \qquad (4)$$

According to the principle of differentiated influence of errors, in order to neglect the initial data error m_s for calculating the total medium error M, M will have to be lower than the measurements errors m_{mas} with more than the precision size m_M :

$$M - m_{mas} \le m_M \qquad (5)$$

According to the relation (5), and to the principle of equal influence of errors, relation (4) becomes:

$$k = \frac{m_{mas}}{\sqrt{M^2 - m_{mas}^2}} = \frac{m_{mas}}{\sqrt{(M + m_{mas})(M - m_{mas})}} = \frac{m_{mas}}{\sqrt{2m_{mas}m_M}} = \frac{1}{\sqrt{2\frac{m_M}{m_{mas}}}} \qquad (6)$$

After Cristescu, 1978, overall average error ratio of precision measurement error M of the denominator of expression (6) must be between 10% ÷ 20%.

Given that errors in high gear must be k times smaller than the measurement errors and considering current gear support network developed in three distinct stages, we can write for the same value of the coefficient of precision (k) relations:

$$m_2 = \frac{m_3}{k}; m_1 = \frac{m_2}{k} = \frac{m_3}{k^2} \qquad (7)$$

By replacing (7) in (1) it is obtained the total square mean error for positioning of a point:

$$M = m_3\sqrt{\frac{1}{k^4} + \frac{1}{k^2} + 1} = m_3 Q \qquad (8)$$

where:

$$Q = \sqrt{\frac{1}{k^4} + \frac{1}{k^2} + 1} \qquad (9)$$

Considering as known the total mean square error of the planimetric positioning of the network points and the coefficient of increase measurement accuracy we can calculate mean square errors of the three stages with the relations:

$$m_3 = \frac{M}{Q} = \frac{0.2N}{Q}, m_2 = \frac{M}{kQ} = \frac{0.2N}{kQ}, m_1 = \frac{M}{k^2Q} = \frac{0.2N}{k^2Q} \qquad (10)$$

where:

N – scale factor;

0.2 mm – graphic positioning precision on plan.

According to the calculated accuracies of the three steps will choose the tools and the work methods.

When performing measurements were used receivers: Stonex S9 GNSS L1, L2 Generation II - Italy, Magellan ProMark 3 L1 and Leica (SR 20 L1) - Switzerland and the method used to perform measurements was the static method.

Geodetic network was comprised of 15 points of which 4 are old points of order V of Romania's national network.

The network drawing with points numbering is shown in Figure 1.

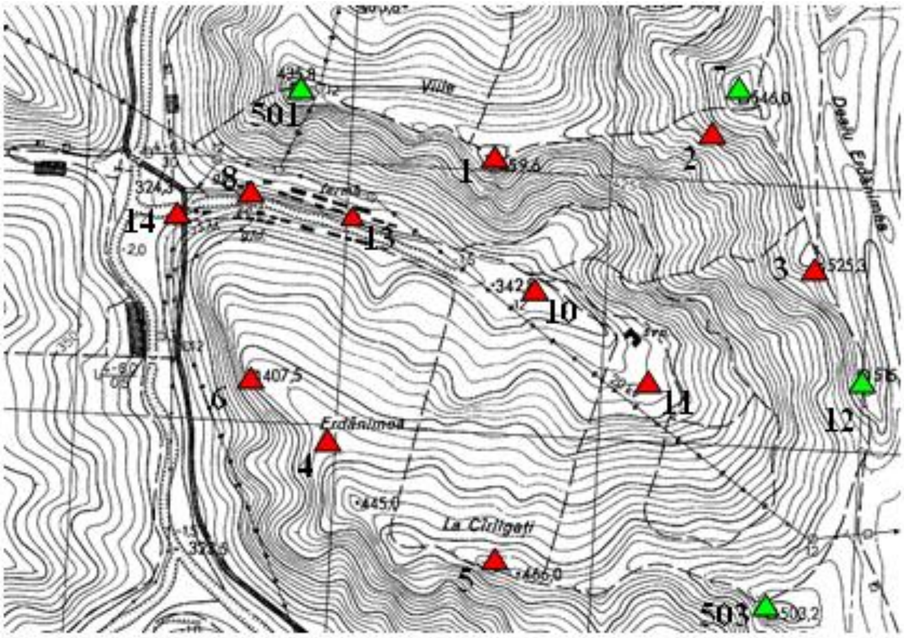

Fig. 1. Network drawing (Δ - points from the old network (501, 7, 12, 503);
Δ - points from the new network)

- The landmarks 501,502 respectively 503,504 were stationed with GPS STONEX GNSS S9 L_1L_2. The same session was interrupted due to low battery, so it appears 2 sets of coordinates for the same point;
- Points 13 and 14 were stationed with GPS MAGELLAN PROMARK 3, whole session uninterrupted for 8 hours;
- Points 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11 and 12 were stationed with GPS LEICA SR 20. Point 8 (9) was stationed 2 times, in order to perform a comparative analysis on it;

Data processing of GPS equipment was achieved with GNSS SOLUTIONS S, starting with organizing on separate files of the raw data collected depending on the model and date of collection of observations.

The best known methods of transformation is Helmert transformation with 7 parameters it is assumed that there are no systematic errors in geodetic networks and there is only transformed linear distortions. For national and local geodetic network is used the Bursa-Wolf model, Badekas-Molodenski model, multiple regression equations method etc. (Neuner 2000, Coşarcă 2003).

Obtaining mode:

- Conversion from the ellipsoidal coordinates on the WGS84 ellipsoid to the geocentric cartesian coordinates;

$$\mathbf{(B,L,h)}_{\mathbf{WGS84}} \rightarrow \mathbf{(X,Y,Z)}_{\mathbf{WGS84}}$$

- Conversion from the Stereographical 1970 coordinates to the ellipsoidal coordinates on the Krasovski 1940 ellipsoid;

$$\mathbf{(x,y)}_{\mathbf{Stereographic\ 1970}} \rightarrow \mathbf{(B,L)}_{\mathbf{Krasovski\ 1940}}$$

- Conversion from the ellipsoidal coordinates (B,L)_Krasovski 1940 and the normal altitude H_MN in the Black Sea 1975 altitude system to the geocentric cartesian coordinates;

$$\mathbf{(B,L,H_{MN})}_{\mathbf{Krasovski\ 1940}} \rightarrow \mathbf{(X,Y,Z)}_{\mathbf{Krasovski\ 1940}}$$

- Direct Helmert transformation.

(X,Y,Z) WGS84 → (X,Y,Z) Krasovski 1940

For determining the parameters it is needed to know the landmarks coordinates in both systems.

The number of landmarks must be more than sufficient in order to make more combinations and to choose the optimal solution. Landmarks must be distributed homogeneously in the work area (Moldoveanu, 2002).

Bursa-Wolf model (Bursa, 1962; Wolf, 1963) is a transformation model which uses seven parameters, for transforming the spatial cartesian coordinates between two geodesic datums (Păunescu *et al.*, 2006)

The transformation implies three translation constants (X , Y, Z), three rotation elements (R_X , R_Y , R_Z) and a scale factor (ΔL). The matrix model can be written like this:

$$\begin{bmatrix} X_{KA} \\ Y_{KA} \\ Z_{KA} \end{bmatrix} = \begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{bmatrix} + \begin{bmatrix} 1+\Delta L & R_z & -R_y \\ -R_z & 1+\Delta L & R_x \\ R_z & -R_y & 1+\Delta L \end{bmatrix} \begin{bmatrix} X_{WGS} \\ Y_{WGS} \\ Z_{WGS} \end{bmatrix} \qquad (11)$$

where:

• $X_{WGS\text{-}84}$, $Y_{WGS\text{-}84}$, $Z_{WGS\text{-}84}$: cartesian coordinates of the geocentric global datum (WGS-84);

• X_{KA}, Y_{KA}, Z_{KA}: cartesian coordinates of the local (KRASOVSKI 1940).

The linear model of the correction equations, in matrix treating through least squares method is written as:

$$\begin{bmatrix} V_x \\ V_y \\ V_z \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 & X_{WGS} & 0 & -Z_{WGS} & Y_{WGS} \\ 0 & 1 & 0 & Y_{WGS} & Z_{WGS} & 0 & -X_{WGS} \\ 0 & 0 & 1 & Z_{WGS} & -Y_{WGS} & X_{WGS} & 0 \end{bmatrix} \begin{bmatrix} \Delta X \\ \Delta Y \\ \Delta Z \\ \Delta L \\ R_x \\ R_y \\ R_z \end{bmatrix} - \begin{bmatrix} X_{KA} - X_{WGS} \\ Y_{KA} - Y_{WGS} \\ Z_{KA} - Z_{WGS} \end{bmatrix} \qquad (12)$$

Solution to determine the unknown parameters is obtained by applying theory of least squares method for solving matrix (Moldoveanu, 2002):

$$X = (A^T P A)^{-1} A^T P L \qquad (13)$$

where:

X - is the vector of unknowns,

A – configuration matrix of coefficients,

L – vector of free terms,

P – observation weighting matrix (in the case of the indirect measurements weighted).

Relative positioning accuracy of common points in both systems, to express the mean square error (standard deviation) calculated by Bessel's formula:

$$s_0 = \sqrt{\frac{V^T P V}{3n-7}} \qquad (14)$$

Bursa-Wolf model has been used successfully in cartesian spatial coordinates transcalculus by some researchers such as Marzooqi (2005) and Noureddine (2006).

RESULTS AND DISCUSSIONS

Measurement errors in three steps calculated using the relation (10) to the total error of positioning M in the 3rd stage are shown in Tab. 1.

Tab. 1

Measurement errors

m_M/m_{mas}	k	Q	m3 [mm]	m2 [mm]	m1 [mm]
10%	2	1.15	34.9	17.5	8.7
20%	1.5	1.28	31.2	20.8	13.9

The results of post processing using GNSS software solutions, and the coordinate values transcalculated in the national system with the software TransDat 4.04 are presented in Tab. 2.

Tab. 2

Post processing using GNSS software

Old landmark	Pct.	B	L	H	X	Y	Z
8	501	46°44'14.88796"N	23°54'01.02622"E	475.759	582571.809	416090.801	435.573
	502	46°44'14.88802"N	23°54'01.02629"E	475.766	582571.811	416090.802	435.58
18	503	46°43'10.98822"N	23°55'27.89919"E	543.169	580573.779	417907.853	503.017
	504	46°43'10.98823"N	23°55'27.89933"E	543.182	580573.779	417907.856	503.03
	1	46°44'05.77519"N	23°54'36.50081"E	499.012	582280.063	416839.864	458.841
	2	46°44'11.11146"N	23°55'18.95622"E	572.74	582432.452	417743.256	532.586
	3	46°43'52.99203"N	23°55'29.80311"E	558.075	581869.946	417965.884	517.925
	4	46°43'39.61157"N	23°53'54.28841"E	446.792	581484.767	415932.639	406.602
	5	46°43'16.71304"N	23°54'35.21797"E	505.227	580765.822	416791.777	465.053
	6	46°43'47.86170"N	23°53'48.74331"E	421.704	581741.1	415818.474	381.512
7L	7	46°44'14.43871"N	23°55'21.49271"E	585.71	582534.436	417798.491	545.557
	8	46°44'01.34752"N	23°53'56.80320"E	371.863	582155.043	415995.355	331.675
	9	46°44'01.33917"N	23°53'56.79560"E	371.819	582154.787	415995.19	331.631
	10	46°43'48.10958"N	23°54'57.03340"E	403.186	581728.711	417268.215	363.023
	11	46°43'37.28589"N	23°55'10.55984"E	414.502	581390.64	417550.79	374.344
11	12	46°43'40.59457"N	23°55'43.81211"E	555.507	581483.186	418258.071	515.362
	13	46°43'58.32263"N	23°54'14.66459"E	378.427	582056.396	416373.191	338.246
	14	46°44'01.38254"N	23°53'41.25037"E	363.62	582160.727	415665.245	323.425
	BACA	46°33'43.40958"N	26°54'43.95540"E	219.193	564260.459	646698.752	185.686
	BAIA	47°39'06.42446"N	23°33'27.75920"E	271.026	684618.168	391774.808	231.482
	DEVA	45°52'42.29505"N	22°54'48.71898"E	246.602	488639.781	338192.35	203.487

The difference between the coordinates of point 8 and 18 obtained in two different periods are shown in Tab. 3.

Tab. 3

Difference between the coordinates of 8 and 18

Point	dx [m]	dy [m]	dz [m]
8 (501)	-0.002	-0.001	-0.007
18 (503)	0.000	-0.003	-0.013

By taking the old coordinates and the coordinates of the same points obtained by post processing to obtain the differences shown in Tab.
4.

Tab. 4

Differences between the old coordinates and the coordinates of the same points obtained by post processing

Point	dx [m]	dy [m]	dz [m]
8	-0.236	0.096	0.224
18	-0.089	-0.083	0.176

There is a difference in the order of decimeters known coordinates of points 8 and 18 triangulation network order V and determined by global positioning. These differences are due to poor accuracy of determination of the points of order V old triangulation network.

CONCLUSION

- It is necessary to verify the accuracy of processing by using different software compensation and adjustment.
- It is recommended to use the nearest permanent stations to reduce the time, in that way we increase the accuracy of the determination of the points that positively affects the productivity and reduces the cost.
- For a better durability in time it is recommended to eliminate the Feno landmarks and replace them with concrete ones.
- Using dual frequency receivers will eliminate some of the errors of the ionosphere by the methods used.

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