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# STUDY ON BUILDING OF PLANIMETRIC NETWORK STAKEOUT FOR A COMMERCIAL SPACE USING COMBINED TECHNOLOGY GPS-TOTAL STATION

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***Abstract***

*This paper presents a case study on the establishment of a support network using modern and high performance technology GPS - Total Station, in which objective was to highlight all the advantages and accuracies results from its implementation on the ground.*
*In the first stage is presented achieving of the support network points using Trimble R6 GPS GNSS, which can reach an accuracy of less than 2 cm, the one he recommends for use in such work. After collecting field data, processing was performed using www.topo-online.ro program, and Trimble Business Center, which led to obtaining a very high efficiency and a high accuracy, which is recorded in tables 5,6,7. For linking to the national coordinate system, points coordinates in the ETRS89 were transformed using software TRANSDAT in Stereographic 1970 coordinate system. Thus network built fully comply all the requirements of accuracy, visibility, accessibility and control, requiring such construction work.*

***Key words:*** *accuracy, coordinates, GPS, total station, support network.*

## INTRODUCTION

Stakeout as the main topographical execution works, intended to implement on the land the characteristic points of construction in plan and height, according to the project. Effectively operation involves preparation of project stakeout respectively calculation of necessary elements and drawing on the field of the axis, contours and details of the project.
Building support networks using artificial satellites represented from the beginning a technical and scientific progress, but over time has undergone a number of improvements, which finally led to increased productivity and increase accuracy absolute position determination points.
The Global Positioning System consists of the following major components (subsystems):

- spatial system (after Paunescu C., 2001, 2012);
- the control system;
- the users system.

The control system consists of several ground located stations for tracking and continuous monitoring satellites. In it is included Operational Control System (OCS), consisting of the main control station, monitor stations and ground control stations.
Orbital elements and ephemeris parameters monitoring and control of this system are:
- $\sqrt{A}$ - square root of the large semi-axis of the orbit; - orbit eccentricity; - $t_{OE}$ - ephemeris time reference; - Io- tilt reference time - I- rate of change of inclination; - $\Omega_O$ -Right Ascension to reference time; - $\varphi$ - change –interest of right ascension; - $\omega$ - argument of perigee; - $M_o$ - mean anomaly at reference time; $\Delta_n$ - average movement changing; ;- Cuc, Cus argument corrections latitude; - Crc, crs- orbital radii corrections; - Cic, Cis - tilt corrections (J. Neuner, 2000 Paunescu C., 2012).
The users include all users of GPS equipment that receives signals for positioning. Ground equipment includes receiver units and auxiliary tools: weather sensors, tribrach, tripods and auxiliary equipment.

## MATERIALS AND METHODS

In the work carried out was used combined method to build support network using GPS technology (GPS Trimble R6) and stakeout the points of detail with very high precision total stations (Leica TCRP 1201 R300, Leica TCR 407 Power). The methods of stakeout the characteristic pints of the building are (Onose D., 2004 and Calina A. et al., 2014): 1 - method of points on alignment; 2 - polar coordinates method; 3- rectangular coordinates method or perpendiculars; 4 - intersection method; 5 - repeated intersection method.

Measurements with GPS receivers are different from those classical because their outcome is either Cartesian coordinates (X, Y, Z) or geographic coordinates - altitude, latitude and longitude. Reference surface is uniform for all those coordinates, which is ellipsoid WGS-84 - World Geodetic System (Calina A., 2013 and Calina Jenica 2012 and 2014). GPS measurements derives from measurements of code (pseudodistances) or phase measurements. The results of these measurements can be processed in real time or post - processing. Measurement is performed in real time when the coordinates are obtained on land from processing a single epoch of measurements or even several epochs. The accuracy of these measurements is small - of meters, so this type of measurement should not be used to determine support networks (J. Neuner, 2000 and Paunescu C. et al., 2012).

The process of determining by post - processing requires observations on magnetic media storage and further processing with an appropriate program. The accuracy of this method depends heavily on the residence time and the distance between points. The residence time is longer, and the distance between points is less, the accuracy of the coordinates increases. The Global Positioning System (GPS) technology has revolutionized terrestrial measurements, leading to radical change in the known criteria design of classical stakeout networks (after Badea Gh. et al., 2001, quoted by Calina A., 2014)

The cause of this development is the advantages offered by GPS technology:

- Points should not have visibility, so that geodetic signals are useless;
- Millimeter instrument precision plus a error range from 1 to 2ppm of the distance between points;
- Increased productivity, resulting lower costs;
- Measurements in all weather conditions (fog, rain, cloudy, day/night);
- Capable of three-dimensional measurements.

The current stage of development for geodetic purposes can be summarized under the following considerations: pretentious requirements of precision ($\sigma \leq \pm 1$cm) can be achieved with ease not only to the specific network stakeout distances (0.3 ... 5km) but for much greater distances. One of the great advantages of this technology is the fact that can design networks very well adapted to the stakeout requirements, whose configuration need not comply classical criteria of the design. Since GPS measurements depend to a very small measure by distance support networks can be achieved with fewer points (distance between points 3-5km), a higher density of points is required into area of the stake objective. After completing the project, following the recognition of land network points will be arranged to ensure:

- measurements favorable conditions (in case of GPS measurement technique)
- free sky for elevation angle 15°;
- not be closer than 200m from broadcasters and 50m high voltage pylons;
- sight between points, which allows operation of stakeout in either method of points construction stake;
- auto transport accessibility to nearby points;
- stability points in time (points to be placed in stable land outside protected areas or pipe networks).

Expected relative accuracy in determining the measured base b can be estimated using the empirical formula (Beutler 1989, 1990)

$$\frac{db}{b} = \sqrt{\frac{1}{2b}}\, mm/_{km}$$

## RESULTS AND DISCUSSIONS

Based on data gathered from the field, later was performed processing of the stakeout network using the program (www.topo-online.ro). It processes data through indirect measurements. In the program has been loaded a text file with a specific form (Figure 1).

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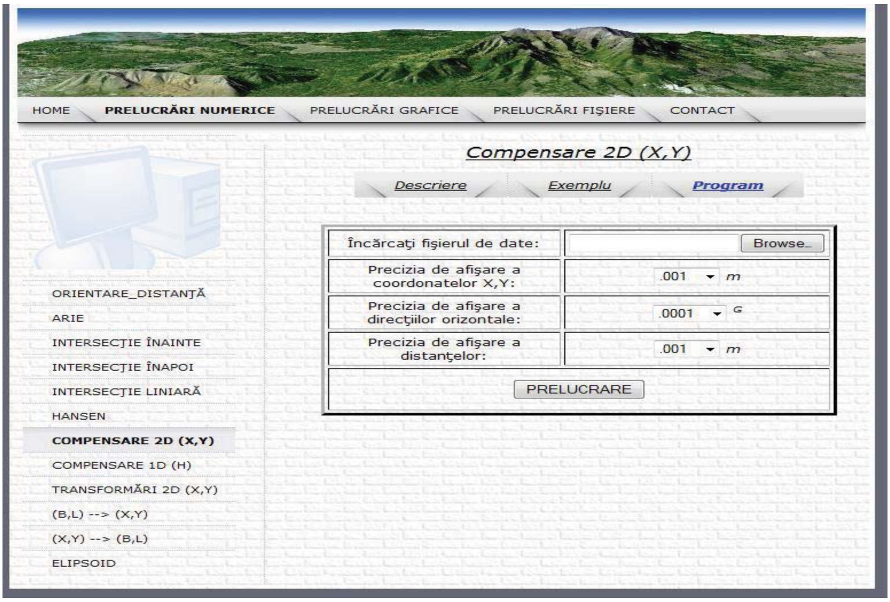


Figure 1. Text file

In order to elaborate the functional model were used provisional values for the unknowns that occur in the model, which refers to the approximate coordinates of the points of the network and also to the provisional values of the orientation angles for the stations at which angular observations were made.

These values must be sufficiently close to the values most likely to be able to waive the terms of second order and higher, from Taylor series expansion. Temporary coordinates chosen for the network points were obtained using GPS technology. The devices used are of the type Trimble R6 GNSS.

Table 1.Technical specifications of the Trimble R6 GPS

| | | | |
|---|---|---|---|
| MEASUREMENTS | GPS: L1 C/A CODE, L2C, L2E (TRIMBLE METHOD FOR TRACKING SIGNAL L2P) GLONASS L1 C/A CODE, L1 CODE P, L2 C/A (ONLY GLONASS-M), L2 CODE P | | |
| DIFFERENTIAL GPS POSITIONING ON CODE | ACCURACY | HORIZONTAL | $\pm(0.25m+1ppm)^2$ RMS |
| | | VERTICAL | $\pm(0.5m+1ppm)^2$ RMS |
| STATIC MEASUREM. | | HORIZONTAL | $\pm(5mm+0.5ppm)^2$ RMS |
| | | VERTICAL | $\pm(5mm+1ppm)^2$ RMS |
| KINEMATIC MEASUREMENT | | HORIZONTAL | $\pm(10mm+1ppm)^2$ RMS |
| | | VERTICAL | $\pm(20mm+1ppm)^2$ RMS |
| AREA OF OPERATION | | TEMPERAT. | from -40$^0$C to +65$^0$C |
| | | MOISTURE | condensation 100% |
| | | IMPERMEABILITY | IP67 for dip depth of 1m |

It was also used data from ROMPOS system, which is based on a National Network permanent GNSS Stations (GPS + GLONASS). Reference stations operate permanent providing data in real time and at predetermined intervals (1h, 24h).

ROMPOS is a position determination system based on GNSS technologies and includes the following services:
- ROMPOS DGNSS - Service for applications in real-time kinematic (positioning accuracy between 3m and 0.5m);
- ROMPOS RTK - service for accurate real-time kinematic applications (up to 2cm accuracy);
- ROMPOS GEO (Geodetic) for postprocessing applications (less than 2cm accuracy).

At work under study was used the method **GPS RTK (Real Time Kinematic)** - Real-time kinematic applications to give an accuracy of up to 2 cm. The measuring range was 1 second, and measurement epochs 300. It states that during measurements was a favorable weather with little wind and temperatures between 15-22°C.

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Calculations were performed using Trimble Business Center. Field measurements have been downloaded from the device as a job, then it was loaded into the program.

Stages of work with Trimble Business Center were:

1. Creation of a project in which to save the results;

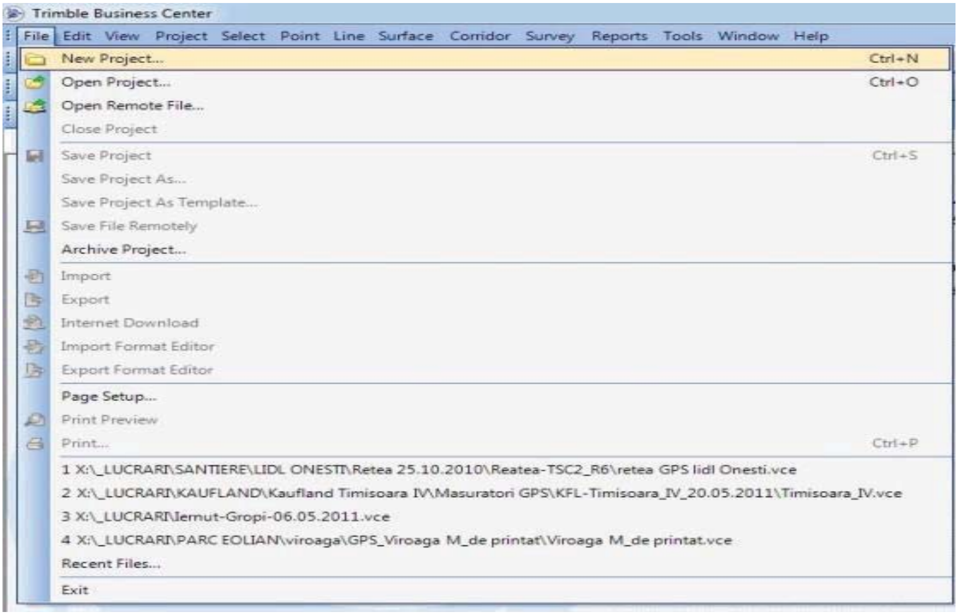


Figure 2. Creation of the project.

2. Importing the job downloaded from the device;

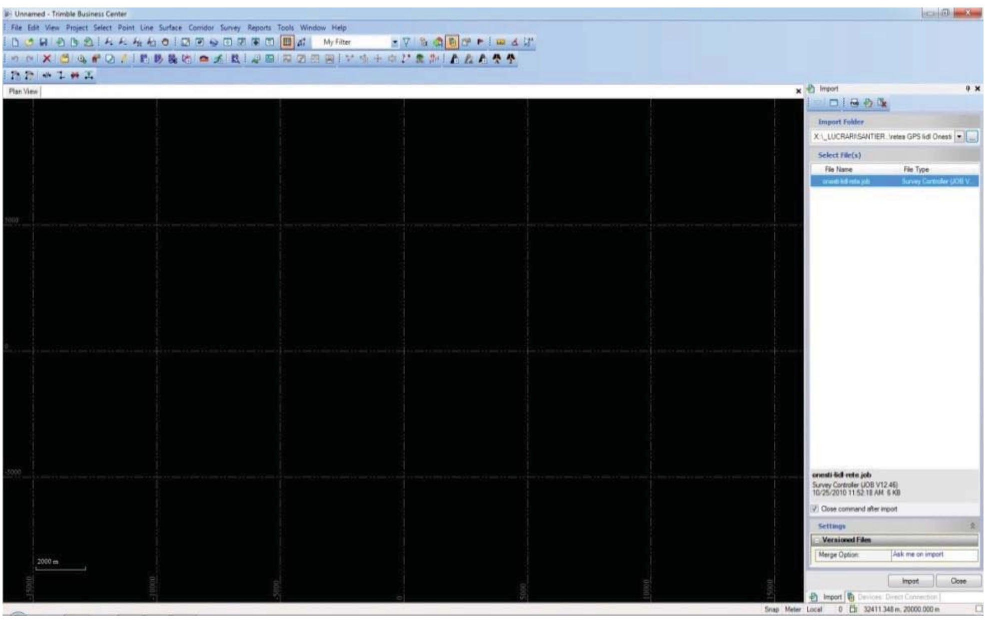

Figure 3. Importing the job

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3. Obtaining GPS network

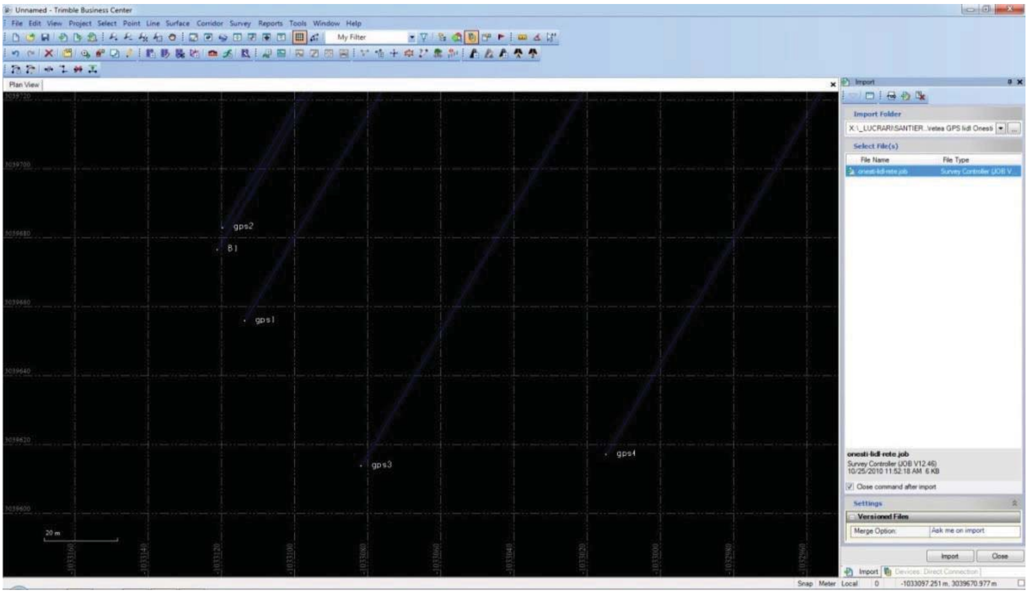


Figure 4. GPS network

4. Exporting data.

They will be saved as a file (.txt). It here will find the coordinates of points in the system ETRS89 in the form: Name point, Latitude (B) Longitude (L) and Ellipsoidal height (He).

Then using software TRANSDAT were transformed point coordinates from ETRS89 in 1970 stereographic coordinate system (system used in Romania).

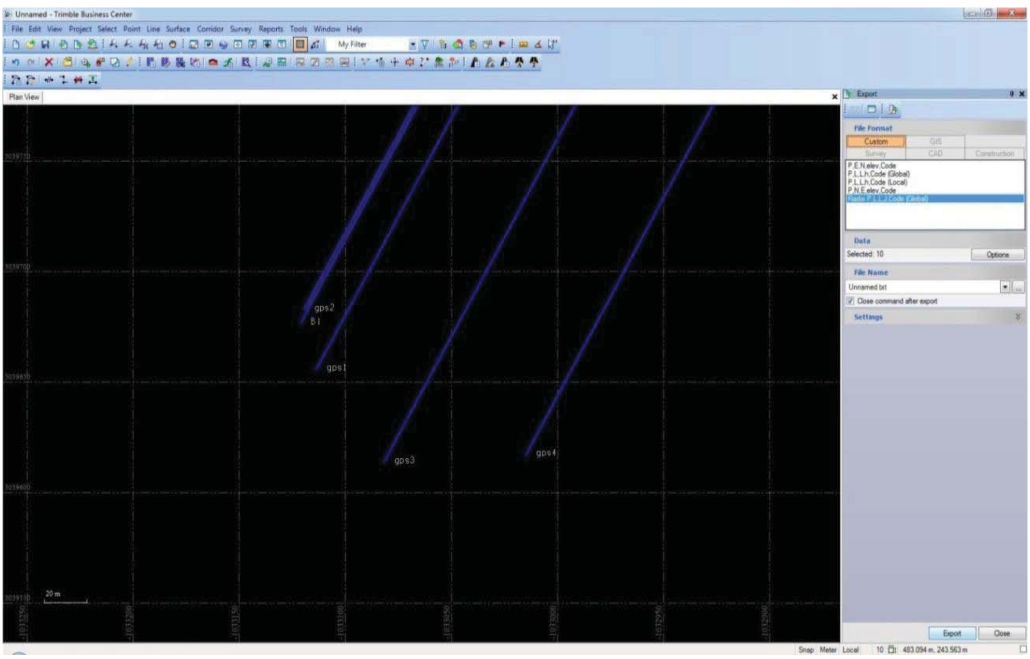


Figure 5. Exporting data to obtain coordinates

The temporary coordinates of the new points are presented in Table 4.

***Data processing for stakeout network of the Commercial Complex***

**1. Initial data**

***Coordinates of the old points***

Table 2. Inventory of coordinates old points

| Name | X [m] | Y [m] |
|---|---|---|
| **1000** | 530383.459 | 636668.591 |
| **1001** | 530597.792 | 636595.661 |

***Directions measured values reduced to the projection plane***

Table 3. Horizontal directions

| Station point | Sight point | Direction [G] |
|---|---|---|
| 1000 | 1001 | 377.3579 |
| | GPS1 | 162.2631 |
| | GPS3 | 148.5953 |
| GPS3 | 1000 | 348.5954 |
| | GPS1 | 342.9416 |
| | GPS2 | 352.3331 |
| | GPS4 | 80.8325 |
| GPS1 | GPS2 | 370.3966 |
| | 1001 | 376.0957 |
| | GPS3 | 142.9483 |
| | 1000 | 362.2564 |

**2. The temporary coordinates of the new thickening points**

Table 4. The temporary coordinates of the new thickening points

| Name | X [m] | Y [m] |
|---|---|---|
| **GPS1** | 530365.739 | 636679.873 |
| **GPS2** | 530389.593 | 636668.697 |
| **GPS3** | 530333.371 | 636718.183 |
| **GPS4** | 530350.560 | 636779.573 |

**3. The result of the processing**

Table 5. Horizontal angular directions offset and their accuracies

| **Station point** | **Sight point** | **Measured direction [G]** | **Corection [cc]** | **Offset direction [G]** | **Standard deviation [cc]** |
|---|---|---|---|---|---|
| **1000** | **1001** | 377.3579 | 19.4 | 377.3598 | ± 12.47 |
| | **GPS1** | 162.2631 | -37.27 | 162.2594 | ± 51.39 |
| | **GPS3** | 148.5953 | 17.87 | 148.5971 | ± 18.88 |
| **GPS3** | **1000** | 348.5954 | -3.82 | 348.595 | ± 18.88 |
| | **GPS1** | 342.9416 | 18.2 | 342.9434 | ± 23.95 |
| | **GPS2** | 352.3331 | -12.26 | 352.3319 | ± 18.19 |
| | **GPS4** | 80.8325 | -2.12 | 80.8323 | ± 20.15 |
| **GPS1** | **GPS2** | 370.3966 | 0.59 | 370.3967 | ± 41.57 |
| | **1001** | 376.0957 | -16.01 | 376.0941 | ± 12.33 |
| | **GPS3** | 142.9483 | -21.2 | 142.9462 | ± 23.95 |
| | **1000** | 362.2564 | 36.63 | 362.2601 | ± 51.39 |

Table 6. Offset distances and their accuracies

| Station point | Sight point | Measured distance [m] | Corection [mm] | Offset distance [m] | Standard deviation [mm] |
|---|---|---|---|---|---|
| **1000** | **GPS1** | 20.89 | -0.77 | 20.889 | ± 3.37 |
| **1000** | **GPS3** | 70.392 | -0.89 | 70.391 | ± 3.54 |
| **1000** | **GPS2** | 6.218 | 1.15 | 6.219 | ± 3.33 |
| **GPS3** | **1000** | 70.392 | -0.89 | 70.391 | ± 3.54 |
| **GPS3** | **GPS1** | 50.178 | 0.68 | 50.179 | ± 3.47 |
| **GPS3** | **GPS2** | 74.921 | -0.13 | 74.921 | ± 3.55 |
| **GPS3** | **GPS4** | 63.878 | 0 | 63.878 | ± 3.52 |
| **GPS1** | **GPS2** | 26.34 | -0.12 | 26.34 | ± 3.39 |
| **GPS1** | **1001** | 246.745 | 9.25 | 246.754 | ± 4.12 |
| **GPS1** | **GPS3** | 50.178 | 0.68 | 50.179 | ± 3.47 |
| **GPS1** | **1000** | 20.892 | -2.77 | 20.889 | ± 3.37 |

Table 7. Offset coordinates of new points and their accuracies

| Point name | | Temporary coordinates [m] | Corections [mm] | Offset coordinates [m] | Standard deviation [mm] | Total standard deviation [mm] |
|---|---|---|---|---|---|---|
| **GPS1** | **X** | 530365.739 | 81.0 | 530365.820 | ± 1.48 | ± 2.01 |
| | **Y** | 636679.873 | -96.0 | 636679.777 | ± 1.37 | |
| **GPS2** | **X** | 530389.593 | 92.0 | 530389.685 | ± 2.06 | ± 2.89 |
| | **Y** | 636668.697 | -78.0 | 636668.619 | ± 2.03 | |
| **GPS3** | **X** | 530333.371 | 36.0 | 530333.407 | ± 1.83 | ± 2.63 |
| | **Y** | 636718.183 | -99.0 | 636718.084 | ± 1.88 | |
| **GPS4** | **X** | 530350.56 | 93.0 | 530350.653 | ± 3.98 | ± 5.51 |
| | **Y** | 636779.573 | 18.0 | 636779.591 | ± 3.80 | |

Table 8. Elements of error ellipses

| Point name | a – semi-major axis [mm] | b – semi-minor axis [mm] | Semi-major axis orientation [G] |
|---|---|---|---|
| **GPS1** | 1.7 | 1.08 | 155.63 |
| **GPS2** | 2.38 | 1.64 | 151.64 |
| **GPS3** | 1.88 | 1.83 | 95.38 |
| **GPS4** | 4.22 | 3.54 | 41.62 |

Table 9. Final points coordinates

| Point name | X(North) [m] | Y(East) [m] | $S_x$ [mm] | $S_y$ [mm] | $S_p$ [mm] |
|---|---|---|---|---|---|
| **1000** | 530383.459 | 636668.591 | --- | --- | --- |
| **1001** | 530597.792 | 636595.661 | --- | --- | --- |
| **GPS1** | 530365.820 | 636679.777 | ± 1.48 | ± 1.37 | ± 2.01 |
| **GPS2** | 530389.685 | 636668.619 | ± 2.06 | ± 2.03 | ± 2.89 |
| **GPS3** | 530333.407 | 636718.084 | ± 1.83 | ± 1.88 | ± 2.63 |
| **GPS4** | 530350.653 | 636779.591 | ± 3.98 | ± 3.80 | ± 5.51 |

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## CONCLUSIONS

Based on studies in the field and in the office, it was concluded that the network support for stakeout axes and the characteristic points of the building must be executed using GPS modern technology and data processing using www.topo-online.ro program, which use as basis for calculation the indirect measurements.

Data collected on the ground had to be compensated, primarily were offset horizontal directions, where corrections are between $2.12^{cc}$ and $37^{cc}$, and the standard deviation of $12.33^{cc}$ and $51.39^{cc}$, then distances were corrections are between 0.12 mm and 9.25mm, standard deviation of 3.33 to 4.12mm.

According to the data processed and calculated was found that the accuracy of determining support network points is high because all the topographic elements have values of standard deviations much smaller than the tolerance allowed in the work of stakeout, for civil and industrial buildings.

In the first phase of the field were determined temporary coordinates of the points of support network after they have been processed and compensated, observing that the standard deviations of X and Y are reduced and total standard deviation does not exceed 5.51mm.

All final coordinates of support network points have been checked and compensated rigorously, which gives us the certainty that their use in stakeout the axes and characteristic points of future construction, will be transmitted without any errors or pressure.

A particularly important phenomenon observed is that surveying and stakeout work efficiency is very high, due to the use of modern and advanced technologies and also staffing costs and decreased significantly compared with the classical method.

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