Practical Measurement of GNSS Technologies from EMLID and SOKKIA Receivers and their Comparison

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Abstract

The practical measurement and comparison of GNSS technologies from SOKKIA and EMLID receivers is only a demonstration of how GNSS surveying technology with accuracy of a few centimetres can be used effectively in conditions of the Czech Republic. Czech surveyors use GNSS technology primarily as a simple data collection in open terrain - it cannot be used in buildings. Most often they are simple measurements such as terrain surveying for the designer, surveying for the purposes of land registry, laying out constructions and others. There are several methods for determination of position, i.e. location in defined reference coordinate and height systems (in the Czech Republic they are S-JTSK and Bpv), according to receiver's software using networked RTK method (corrections from providers of network of reference stations - such as TOPNET, CZEPOS, and others) or RTK base – rover method or fast static method. The task is to compare the measurement time intervals from each technology and the accuracy of the measurement on the known points of the Czech national trigonometric network. At the end the financial comparison of both technologies is presented.

Keywords: *GNSS, comparison of accuracy, Emlid, Sokkia*

1. Introduction

This paper discusses the testing of various GNSS instruments from two different manufacturers, namely SOKKIA of Japan, which was established in 1920 and its first product was a dual-axis spirit level, and in 1997 the first RTK GNSS technology was produced [1]. The second brand is EMLID from Hungary, which was founded in 2014 as a project of two students who raised money from sponsors to develop their first GNSS instrument called Reach in 2015 [2].

Testing is taking place on the Czech Republic's point array, which has been built since the 1920s. At the same time, the cartographic cone mapping Křovákovo, the coordinate system JTSK (Unified Trigonometric Cadastral Network System) and the height system Bpv (Balt after leveling) are also being defined [3]. These are the points that form the positional basis of the Czech Republic, namely trigonometric points and thickening points. The coordinates of these points were created by triangulation - measuring angles with theodolites. However, the advantage of some of the densification points used for our testing of GNSS instruments is that in 2008 these points were supplemented with ETRS89 coordinates, so we know that the coordinates have been refined or newly determined by GNSS measurements [4]. To be able to measure in S-JTSK and Bpv, it is necessary to transform to these two systems from the ETRS89 system, in which the coordinates are determined by GNSS equipment using the network of permanent stations in the Czech Republic. Article by Dr. Nágl, Dr. Řezníček from the journal GAKO No. 10/2018, who discuss the transformation between these two systems [5].

How to test GNSS technologies has already been proven by colleagues from Slovakia, namely Vladimír Sedlák, who compared SOKKIA static measurements with measurements using a total station [6]. Similar testing appears in this article only on a smaller scale. Developments in technology around the world are allowing surveyors to use measurement methods such as simultaneous angle and length measurement, GNSS, unmanned aerial vehicles (UAVs), laserscanning and Lidar technologies. This testing focuses exclusively on GNSS technologies, the applications of which were introduced in the 1990s [7], using differencing methods - determining base-to-rover coordinate differences. At the beginning of the 21st Century, the RTK (Real Time Kinematic) method was introduced, which uses GNSS signals for real-time differential measurements with centimeter accuracy [8]. In their short study, authors Spangero and Papadimitratos from the University of Sweden tested the quality of RTK measurements and the effects and interference on reference stations [9]. Their calculations are performed in RTKLib, which is a software used, among other things, for RTK measurements, but also for processing measurements by the static method. RTKLib is also used for Emlid Studio, which was used to calculate the static data for this paper [10].

However, evaluation of the measurement reliability of various other inexpensive RTK-GNSS tags that have been tested in outdoor, urban, and into city measurements has shown that they cannot maintain a fixed integer solution for the solution time in dynamic applications [11]. Other GNSS testing of low-cost apparatus has shown that these technologies can be used. More specifically, Nguyen and Cho from Japan [12]. Another interesting test of low-cost GNSS technology, but now with an IMU (Inertial Measurement Unit) was carried out by Cahyadi and team for autonomous ship navigation, who achieved an accuracy of approx. 0.3 m in position [13]. As a result, this paper compares low-cost geodetic GNSS instruments from EMLID, namely the Reach RX and RS3 variants, with geodetic instrument variants from established companies such as SOKKIA, namely the GRX3 and GCX3 models. The aim of the paper is to show that even low-cost technology is applicable to practical surveying and is able to compete with established solutions from other manufacturers.

2. Method

The measurements were carried out at 14 points evenly distributed throughout the Czech Republic. A total of 4 different GNSS instruments and several methods were used, namely fast static method, networked Real time kinematic using network corrections and Real time kinematic using base - rover solution.

2.1 GNSS EMLID Reach RX

RTK GNSS technology, which has the advantage of its weight and ease of use. It does not use the TILT function and thus works only on the need to hold the rod vertically. Its accuracies are RTK H: 7 mm + 1 ppm and V: 14 mm + 1 ppm. It collects GPS, GLONASS, BeiDou and Galileo satellite systems and operates at 5Hz [14].

Fig. 1 GNSS Emlid RX, photo by Ondřej Váňa

2.2 GNSS EMLID Reach RS3

GNSS technology, which has a tilt of more than 60° compared to RX and allows measurements not only in RTK mode, but also fast static method, BASE - ROVER using LoRa/UHF and PPK. The tilt is an IMU unit that has no magnetic field influence. Its accuracies in RTK are the same as RX. Furthermore, in RTK with IMU at inclination more than 60° the accuracy is degraded by up to 2 cm. In the fast static measurement mode it has accuracies of H: 4 mm + 0.5 ppm and V: 8 mm + 1 ppm and operates at up to 10 Hz [15].

Fig. 2 GNSS Emlid RS3, photo by Ondřej Váňa

2.3 GNSS SOKKIA GRX3

GNSS technology, which compared to the RS3 has a limited tilt, only up to 15° to one side, and includes a Compass, which is necessary to calibrate once in a while, i.e. that there is an influence of the magnetic field. The technology can be used similarly to RS3, i.e. fast static method, BASE - ROVER, via Bluetooth and UHF. Its accuracies are in RTK H: 5 mm + 0.5 ppm and V: 10 mm + 0.8 ppm. For inclinations greater than 10°, the measurement deteriorates by up to 2 mm. In the fast static method mode, the accuracy is H: 3 mm + 0.4 ppm and V: 5 mm + 0.5 ppm and works up to 20Hz [16].

Fig. 3 GNSS SOKKIA GRX3, photo by Ondřej Váňa

2.4 GNSS SOKKIA GCX3

RTK GNSS technology, which is very similar to Emlid's RX technology. It has no tilt and thus only works on vertical measurements. No less it can be licensed to measure fast static methods, and can possibly be used on a long range Bluetooth BASE - ROVER. Its RTK accuracies are H: 10mm + 0.8 ppm and V: 15mm + 1.0 ppm and it works up to 10 Hz [17].

In the results you will find abbreviations such as dY, dX and dZ, which are the coordinate differences between the known and measured point. The dP value is then the positional deviation, which was calculated using the formula:

$$dP = \sqrt(dX\text{^}2 + dY\text{^}2) \quad (1)$$

Fig. 4 GNSS SOKKIA GCX3, photo by Ondřej Váňa

Points that have failed the internal testing of the required accuracy of the control of the current point are highlighted in red. Each point of the Czech Republic point field has its own accuracy criterion testing. The points of the basic point field have a mean coordinate error:

$$Mxy = 0{,}015\ m \tag{2}$$

and since a positional limit deviation is needed, it was calculated by the formula:

$$MP = 2 * \sqrt{2} * Mxy = 0{,}042 \text{ m} \tag{3}$$

The same is true for points in the compaction point field that were also measured using a non-tested GNSS technology and have a mean coordinate error:

$$Mxy = 0{,}020\ m \tag{4}$$

and since a positional limit deviation is needed, it was calculated by the formula:

$$MP = 2 * \sqrt{2} * Mxy = 0{,}057 \text{ m} \tag{5}$$

The same applies to the determination of the height accuracy of a trigonometric point, which has a specified height of 0.1m, namely:

$$M\_z = 0{,}10\ m \tag{6}$$

and since a positional limit deviation is needed, it was calculated by the formula:

$$Mz = 2 * \sqrt{2} * M\_z = 0{,}282 \text{ m} \tag{7}$$

The same applies to the determination of the height accuracy of a compaction point, which has a specified height of 0.1m, namely:

$$M\_z = 0{,}05\ m \tag{8}$$

and since a positional limit deviation is needed, it was calculated by the formula:

$$Mz = 2 * \sqrt{2} * M\_z = 0{,}141 \text{ m} \tag{9}$$

The points that exceeded the values of 42mm and 57mm were marked in red in the testing. In addition, the rows that are in light yellow show the calculation of the fast static method in FLOAT mode, the rows that have no real solution calculations are shown in light red, and the rows that have no data are shown in light purple.

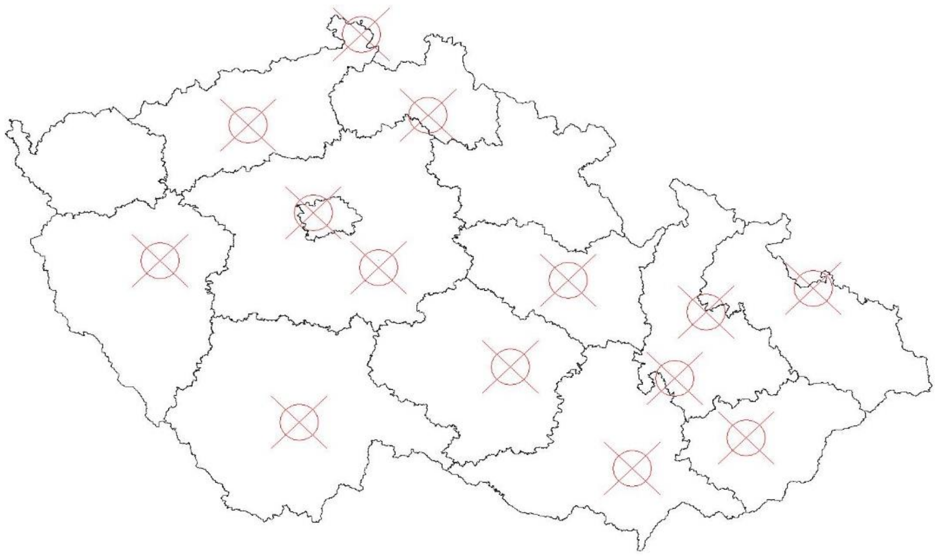


Fig. 5 Map of Czechia where was testing GNSS measure

The following graphs discuss the actual measurement and testing of the data.

Tab. 1 Specified default coordinates to the point Cerekvice nad Loučnou

Location:	*Cerekvice nad Loučnou, Pardubický kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
924082061	617371,63	1079717,16	291,37	2008, GPS methods	20 **(57)**

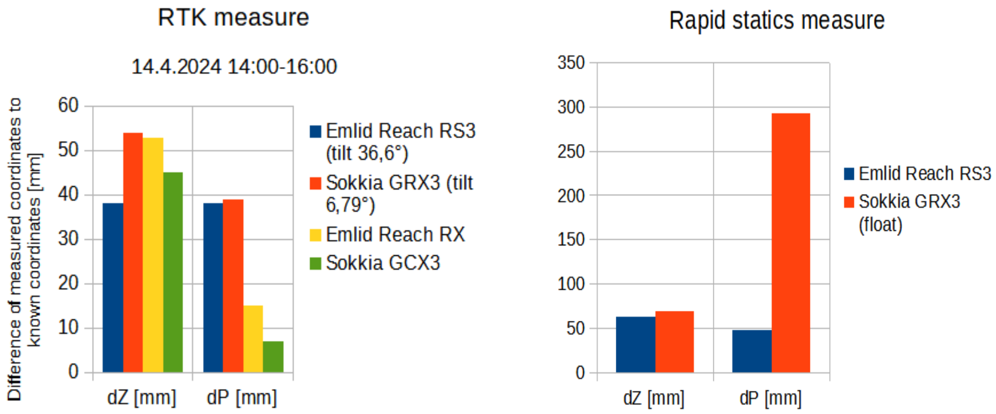


Fig. 5 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Cerekice nad L.

Tab. 2 Specified default coordinates to the point Kostelec na Hané

Location:	*Kostelec na Hané, Olomoucký kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
934080280	564886,66	1128573,81	305,89	1939, trigonometry	15 **(42)**

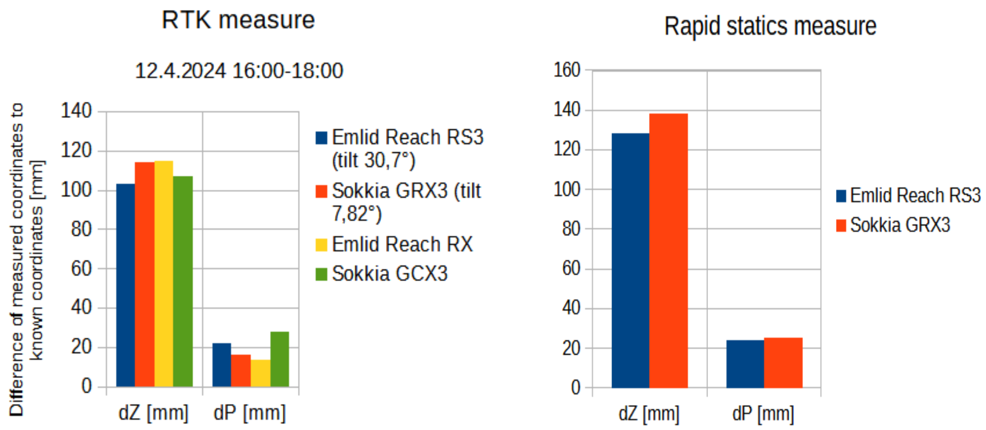


Fig. 6 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Kostelec na Hané

Tab. 3 Specified default coordinates to the point Kyjov

Location:	*Kyjov, Vysočina*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
933232130	647778,779	1127480,733	617,885	2008, GNSS technology	20 **(57)**

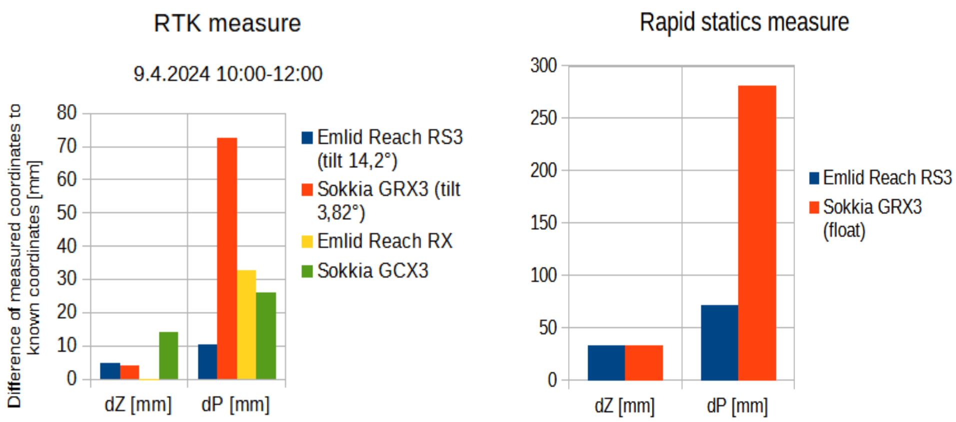


Fig. 7 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Kyjov

Tab. 4 Specified default coordinates to the point Lišov

Location:	*Lišov, Jihočeský kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
941212320	747548,656	1158768,04	505,049	2008, GNSS technology	20 **(57)**

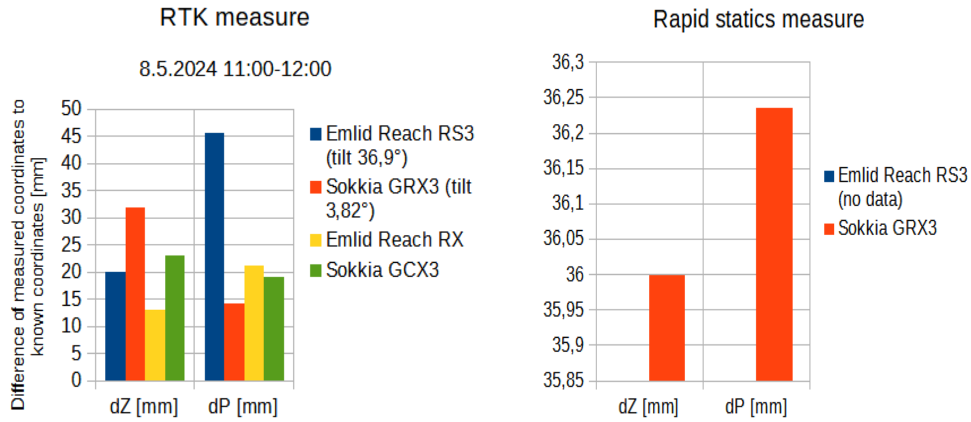


Fig. 8 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Lišov

Tab. 5 Specified default coordinates to the point Želenice

Location:	*Želenice, Ústecký kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
906192480	783404,646	988462,099	206,781	2008, GNSS technology	20 **(57)**

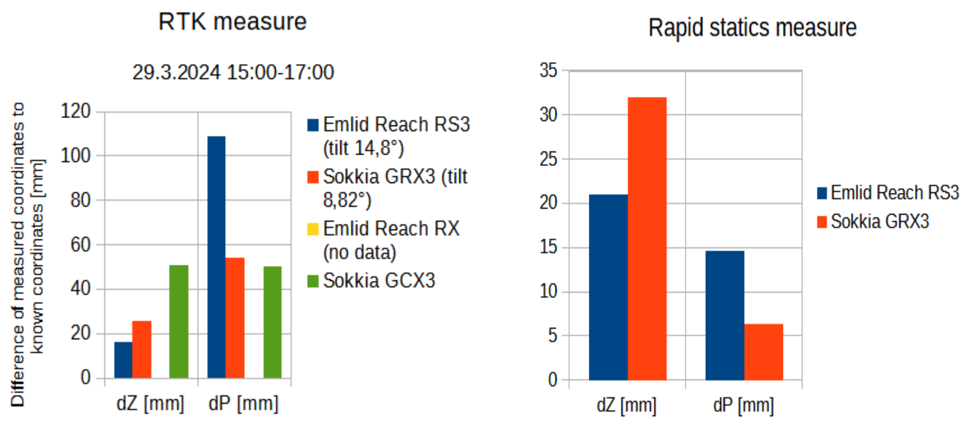


Fig. 9 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Želenice

Tab. 6 Specified default coordinates to the point Oldřišov

Location:	*Oldřišov, Moravskoslezský kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
927242160	493583,564	1084090,37	296,436	2008, GNSS technology	20 **(57)**

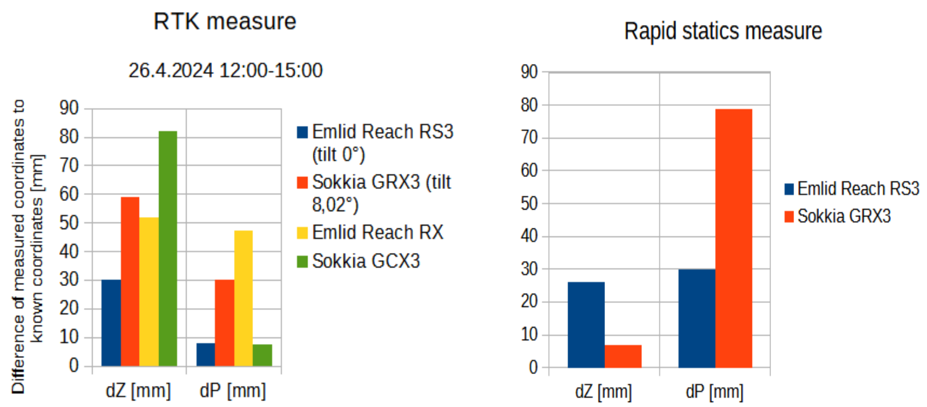


Fig. 10 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Oldřišov

Tab. 7 Specified default coordinates to the point Plzeň

Location:	*Plzeň, Plzeňský kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
920123230	825698,54	1069070,617	320,15	2008, GPS methods	20 **(57)**

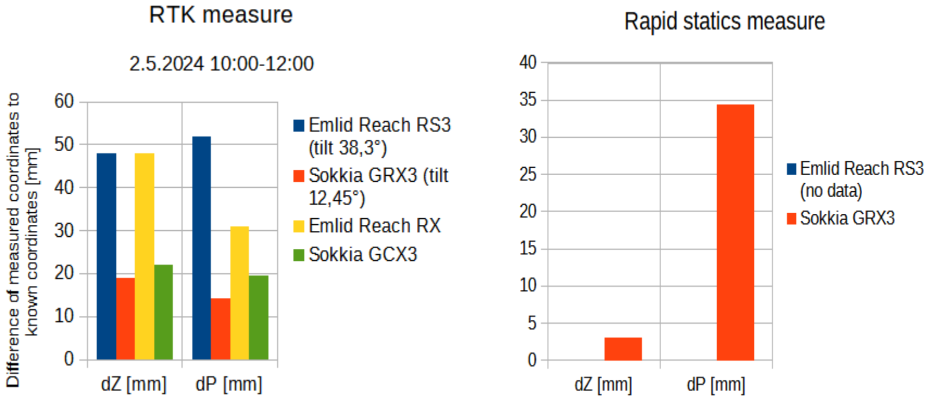
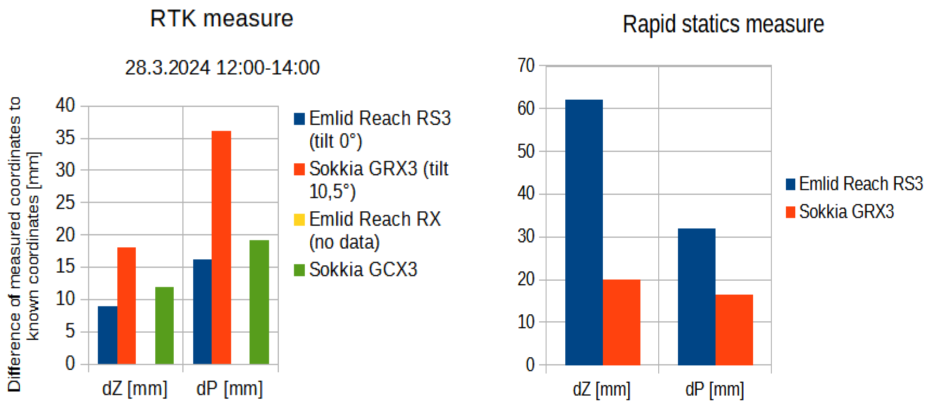


Fig. 11 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Plzeň

Tab. 8 Specified default coordinates to the point Ladronka

Location:	*Ladronka, Praha*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
914251370	747480,667	1043340,246	363,813	2008, GPS methods	20 **(57)**

Fig. 12 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Praha



Tab. 9 Specified default coordinates to the point Pěnčín

Location:	*Pěnčín, Liberecký kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
908192460	688852,963	989914,632	403,15	2008, GPS methods	20 **(57)**

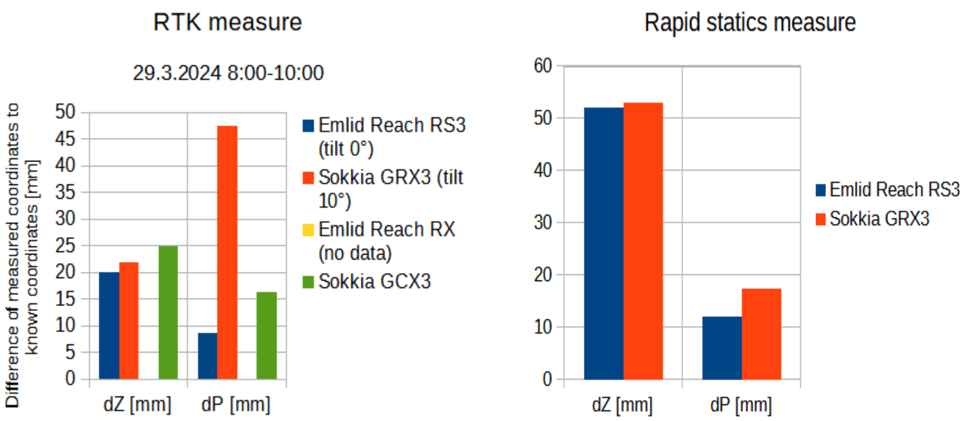


Fig. 13 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Pěnčín

Tab. 10 Specified default coordinates to the point Staré Křečany

Location:	*Staré Křečany, Ústecký kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
901152230	723060,73	945810,5	426,171	2009, GPS methods	20 **(57)**

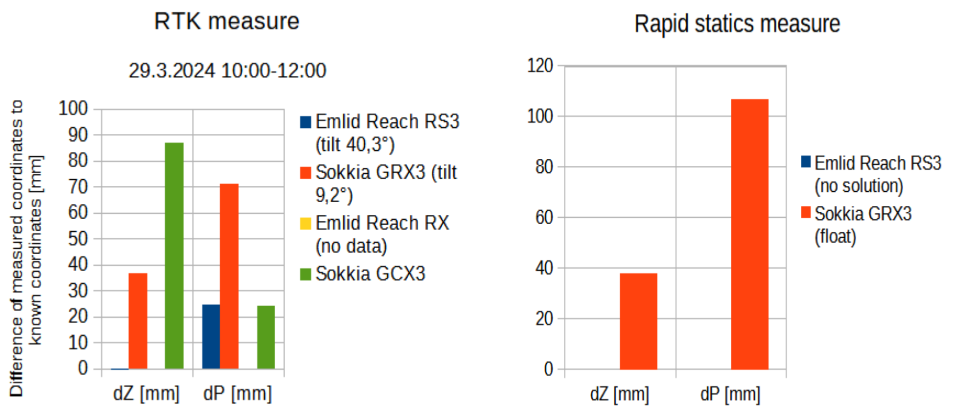


Fig. 14 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Staré Křečany

Tab. 11 Specified default coordinates to the point Zaječí

Location:	*Zaječí, Jihomoravský kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
944250290	591726,8	1197783,59	261,86	1978, trigonometry	15 **(42)**

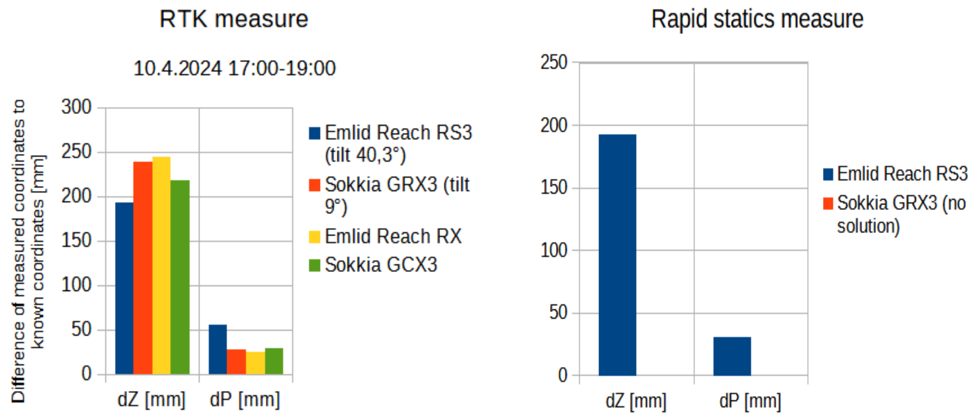


Fig. 15 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Zaječí

Tab. 12 Specified default coordinates to the point Tečovice

Location:	*Tečovice, Zlínský kraj*			THE SPECIFIED COORDINATES	
Point	**Y [m]**	**X [m]**	**Z [m]**	**point origination**	**Accuracy [mm]**
945122240	526530,93	1165589,125	206,189	2008, GPS methods	20 **(57)**

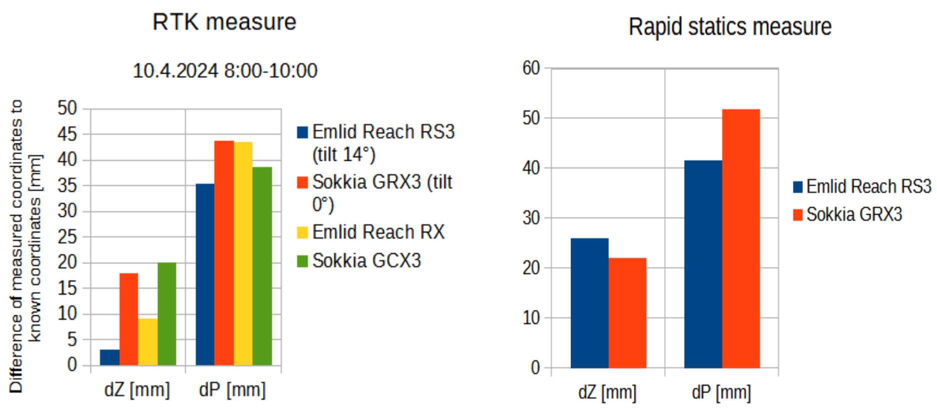


Fig. 16 Graph showing coordinate differences and heights relative to the starting coordinates of the point in Tečovice

The sample standard deviations of the individual coordinates are calculated from the tabulated dY, dX, and dZ, respectively, using the following relation:

$$m_Y = \sqrt{\frac{\sum dY^2}{n-1}} \tag{6}$$

$$m_X = \sqrt{\frac{\sum dX^2}{n-1}} \tag{7}$$

$$m_Z = \sqrt{\frac{\sum dZ^2}{n-1}} \tag{8}$$

From these, the sample coordinate standard deviation:

$$m_{XY} = \sqrt{\frac{m_Y^2+m_X^2}{2}} \tag{9}$$

The following Tab. represents the resulting Mxy for each receiver according to the quality of the points:

Tab. 13 RTK Receiver the value Mxy

Receiver RTK	**trigonometric points (0,042m)**	**compaction points (0,057m)**
EMLID Reach RS3	0,042 m	0,031 m
EMLID Reach RX	0,020 m	0,022 m
SOKKIA GRX3	0,022 m	0,034 m
SOKKIA GCX3	0,029 m	0,018 m

All measurements met the required accuracy within the tolerance limits. The Emlid Reach RS3 technology stood out the most, because it includes measurements with a larger angle of inclination than allowed by, for example, the Sokkia GRX3. The most interesting finding, however, was the Emlid Reach RX receiver, which, given its cheap purchase price, more than held up well with its results.

The following Tab. represents the resulting Mz for each receiver according to the quality of the points:

Tab. 14 RTK Receiver the value Mz

Receiver RTK	**trigonometric points (0,282m)**	**compaction points (0,141m)**
EMLID Reach RS3	0,219 m	0,035 m
EMLID Reach RX	0,270 m	0,041 m
SOKKIA GRX3	0,264 m	0,036 m
SOKKIA GCX3	0,242 m	0,046 m

The height measurements in the compaction point field met the specified difference value of 5cm. This is mainly due to the fact that these points were surveyed in 2008 using the GPS fast static method and therefore do not differ that much from each other. Meanwhile, whereas the heights in the trigonometric point field were determined trigonometrically, i.e. computed, the error here is much larger. Even so, all RTK measurements fit within the specified limit.

Next, it is checked that 60% of the differences of the dP position determination are less than √2 m_XY. And the same for the dX and dY components (60% of which should be less than m_XY). Considering the fact that

$$m_{XY}^2 = \frac{m_Y^2+m_X^2}{2} \tag{10}$$

and if $m_Y \approx m_X \approx m_C$, then

$$m_{XY}^2 = \frac{m_C^2+m_C^2}{2} = \frac{2m_C^2}{2} = m_C^2 \tag{11}$$

The length is calculated from the coordinates using Pyhtagoras' theorem:

$$s^2 = \left(X_i - X_j\right)^2 + \left(Y_i - Y_j\right)^2 \tag{12}$$

We apply the law of accumulation of standard deviations

$$(2s)^2 m_s^2 = \left[2\left(X_i - X_j\right)\right]^2 m_{X_i}^2 + \left[-2\left(X_i - X_j\right)\right]^2 m_{X_j}^2 + \left[2\left(Y_i - Y_j\right)\right]^2 m_{Y_i}^2 + \left[-2\left(Y_i - Y_j\right)\right]^2 m_{Y_j}^2 \tag{13}$$

$$s^2 m_s^2 = \left(X_i - X_j\right)^2 m_{X_i}^2 + \left(X_i - X_j\right)^2 m_{X_j}^2 + \left(Y_i - Y_j\right)^2 m_{Y_i}^2 + \left(Y_i - Y_j\right)^2 m_{Y_j}^2 \tag{14}$$

Then, if $m_Y \approx m_X \approx m_C$, then:

$$\boxed{s^2 m_s^2 = \left(X_i - X_j\right)^2 m_C^2 + \left(X_i - X_j\right)^2 m_C^2 + \left(Y_i - Y_j\right)^2 m_C^2 + \left(Y_i - Y_j\right)^2 m_C^2} \tag{15}$$

And from that

$$s^2 m_s^2 = 2m_C^2 \left[\left(X_i - X_j\right)^2 + \left(Y_i - Y_j\right)^2\right] \tag{16}$$

$$s^2 m_s^2 = 2m_C^2\, s^2 \tag{17}$$

$$m_s^2 = 2m_C^2 \tag{18}$$

And since we know from before že $m_{XY}^2 = m_C^2$ then

$$m_s^2 = 2m_{XY}^2 \tag{19}$$

When testing, we calculate the length differences Δ

$$\boxed{\Delta = k - s} \tag{20}$$

Where k is the length measured by the band, or total station, s is the length determined from the GNSS measurement
Law of accumulation of standard deviations

$$m_\Delta^2 = m_k^2 + m_s^2 \tag{21}$$
$$m_\Delta^2 = m_k^2 + 2m_{XY}^2 \tag{22}$$

where after m_k we add the accuracy of length measurement by the total station or band, for simplicity we consider the length measurement by these technologies as the same and after m_XY we add the coordinate standard deviation from the previous section. The accuracy of the length measurement was set as 3mm per 1000m. If we then use the length differences Δ to calculate the sampling standard deviation of the length difference

$$\underline{m_\Delta^2} = \sqrt{\frac{\sum \Delta^2}{n}} \tag{23}$$

so we can compare it to m_Δ^2. Below is a Tab. of comparison values.

Tab. 15 Comparison of the quality of the measured values by the base-rover method versus the measured length by other technologies

Receiver RTK	$\boxed{\Delta}$	$\underline{m_\Delta^2}$
EMLID Reach RS3	0,023 m	0,052 m
EMLID Reach RX	0,023 m	0,031 m
SOKKIA GRX3	0,010 m	0,041 m
SOKKIA GCX3	0,010 m	0,034 m

Significantly smaller _m_Δ^2 indicates that in relative positioning (between the base and rover to tens of meters) the apparatus are more accurate than in positioning in a binding coordinate system, where the differences also reflect the accuracy of the starting point determination. In this case, the accuracy of the resulting data was more likely to be consistent with the RTK measurements.

3. Conclusion

As it can be noticed, the coordinates that were obtained by GNSS technologies and the coordinates that were obtained X years ago by another technology available at that time do not differ in most cases and, on the contrary, they meet the assumptions of this test to verify cheap GNSS technologies. Another verification parameter was the computed vectors between detailed points that were surveyed using a total station or a tape measure.

The calculation of the fast static method was different for each measurement point. Often all satellite system providers were used, sometimes only GPS Navstar had to be switched and sometimes only Glonass and BeiDou. For the calculation, primarily navigation messages from the TopNet service provider were used, in places where it was not possible to perform the calculation, messages from CZEPOS were used. This may be due to the fact that the vector for calculating coordinates and altitude was too long (distance of the reference station - rover) and the measurement record too short. The average measurement time of the fast static method was 15-20minutes at a given point.

Exceeding the deviation limits for RTK measurement points may be caused by temporary overlapping by another object, not monitoring the statistical values of HRMS (positional deviation) and VRMS (height deviation) during the measurement, or PDOP values that control the quality of our field measurements. For the points No. 944250290 (Zaječí), No. 934080280 (Kostelec na Hané) the differences in heights are more than 15 cm for all methods. This is due to the fact that the coordinates that these points have were obtained by the Trigonometric Determination method over large distances, so the height was determined inaccurately. In the case of point No. 926252440 (Paseka), the question is whether the point, on the other hand, has not shifted in height during its existence. Next, the measured vectors were compared, i.e. the distances were calculated from the coordinates of the measured points and compared to the measured total station / rangefinder in the field. The results showed that cheap GNSSs do not deviate even from these measurement deviations, see testing above.

Based on this testing, it can be concluded that the technology, which costs around 2,500Euro (Emlid RX) or 3,100Euro (Emlid RS3), can be fully used for surveying activities in practice. Their comparator in this test was the SOKKIA technology, which financially is almost 2-3 times more expensive. Outside of this test, testing has also been conducted with other brands that also have IMU units, resulting in very similar results.

This takes surveying and the use of GNSS technology to a whole new dimension. Just look at the fact that e.g. iOS is attempting LiDAR on mobile phones, to which GNSS Emlid can be connected, and we have a point cloud, admittedly less accurate than mobile scanners, but taken in S-JTSK and Bpv. It is only a matter of time before practitioners start to see GNSS as an essential accessory that every surveyor, builder, architect etc. can have for measurement and data collection purposes.

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