Comparison of GNSS, Laser Scanning, and Total Station Measurement Accuracy with Criteria of Digital Technical Map in Czechia

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Abstract

Criteria of the Digital technical mapping (DTM) in Czechia requires horizontal accuracy of 14 cm and elevation accuracy of 12 cm. This study compares GNSS, total station, and mobile laser scanning measurement techniques to evaluate their precision against these legal standards. GNSS is a fast and effective positioning tool but can be affected by satellite position, signal interference, and environmental factors. To test its accuracy, GNSS measurements were compared to total station data, which was aligned with the national geodetic control network. Independent verification was carried out using mobile laser scanning, which also produced sample outputs for DTM evaluation. Results show that GNSS achieves reasonable accuracy in favorable conditions but remains sensitive to external influences. The total station provides the highest precision due to stable reference points and controlled measurement techniques. Mobile laser scanning offers rich spatial data but requires extensive processing and may have accuracy challenges in complex terrain. The generated point cloud serves as a reference dataset for terrain modeling in digital technical mapping. Each method has strengths and limitations. GNSS is useful for rapid data collection, total stations ensure high accuracy, and laser scanning supports detailed spatial documentation. This study helps refine measurement strategies for digital technical mapping while meeting Czech legal accuracy requirements.

Keywords: GNSS surveying, Digital technical Map of Czechia (DTM), Total statin surveying, Mobile laser scanning (MLS), GNSS IMU accuaracy without limit, 4 NTRIP casters

1. Introduction

This scientific paper examines the measurement accuracy of three commonly used geospatial technologies—total station, GNSS, and mobile laser scanning—within the context of standards defined for the Digital Technical Map (DTM) in the Czech Republic, as introduced by Decree No. 393/2020 Sb [1], effective from 2024.

These technologies are not unconventional, especially total stations, which are considered standard in professional surveying. This study uses instruments by Sokkia, essentially equivalent to equipment manufactured by Topcon. Similar devices have been used, for instance, in calibration procedures of measurement bases [2]. Another example of total station applications is found in archaeology, where they are employed for documenting historical developments of archaeological sites [3].

GNSS, widely adopted beyond the field of geodesy, remains one of the most frequently used technologies for data collection in open environments. However, GNSS measurements are still strongly influenced by solar activity and ionospheric interference as of 2025, which may have affected the data presented in this study. A study conducted by researchers from China and Australia discusses how to manage ionospheric uncertainties during ionospheric scintillations within RTK network-based methods [4]. Additionally, research conducted in Tokyo addresses the improvement of GNSS data accuracy in urban environments and mitigation of multipath effects [5]. GNSS is not limited to RTK methods; advanced techniques such as Precise Point Positioning (PPP) also enable real-time position and elevation estimates with decimeter-level accuracy, independent of RTK networks. This has been investigated in a study that, aside from PPP methodology, reviews the European GNSS market [6].

The European satellite system Galileo has long been in use alongside GNSS and has demonstrated certain benefits. A study from a Turkish university concludes that Galileo substantially improves the reliability of position fixing [7]. This paper also includes a comparison of different GNSS measurement results obtained using various RTK correction providers. In the Czech Republic, several correction services are available, including the governmental provider CZEPOS and private services such as TopNET, Geoorbit, and VRSnow. This subject has already been partly analyzed in a publication from the Czech Technical University [8].

Mobile laser scanning (MLS) has also become a widely adopted method in the field. It owes its practicality to SLAM (Simultaneous Localization and Mapping) technologies, originally developed by the now-defunct company GeoSLAM, currently integrated into the FARO technology portfolio [9]. A related study investigated the influence of the number of georeferencing targets on the accuracy of point clouds obtained via MLS [10]. The findings show that when the spacing between targets exceeds 100 meters, accuracy deteriorates. In this paper, the distance between reference points was maintained under 50 meters.

All measurements for this study were conducted by a single skilled operator familiar with the precise locations to be surveyed, ensuring consistency in identifying and extracting reference points from the point cloud. The objective of this paper is not to evaluate differences among surveyors, but rather to assess the measurement capabilities of individual technologies and determine whether they meet the required accuracy criteria for inclusion in DTM datasets.

The comparison is based on the specifications outlined in Appendix No. 2 of Decree No. 393/2020 Sb [1], where positional accuracy is classified into five categories. According to the current regulation, newly collected data are automatically assigned to class TP3, which mandates a maximum mean coordinate error of mxy = 14 cm and a mean elevation error of mz = 12 cm.

The aim of this article is to metrologically compare the quality of datasets acquired using independent technologies against the accuracy classes defined for the DTM. In its final section, the article presents examples of data deliverables produced by each method and proposes standardized formats suitable for submission into the official records of the Digital Technical Map system.

2. Methods and Materials

Three measurement methods were selected based on their suitability for data collection within the framework of the Digital Technical Map (DTM) and the fact that none of them require special authorization for use. For the measurement campaign, a location with a sufficiently dense network of state geodetic control points was chosen, which enabled precise orientation for the total station measurements. In addition, the nearest fixed reference station for GNSS measurements was located more than 18 kilometers away, providing favorable conditions for conducting multiple independent GNSS observations.

The point cloud generated by the mobile laser scanning was georeferenced using control points previously surveyed by independent GNSS measurements carried out in RTK mode using virtual reference stations. The selected location for the study was the Church of Saint Catherine in the village of Jeníkov, Benešov District, Central Bohemian Region.

- GNSS Nivel Point 3

In the first half of 2024, the company 3gon Positioning introduced a new GNSS technology featuring an integrated IMU unit, allowing measurements to be performed without significant tilt limitations. This high-performance receiver is capable of surveying up to 95 percent of a given object and receiving signals from as many as 1408 satellites, which substantially enhances the precision and reliability of the measurements [11]. Although the software supporting this device remains under development, essential functionalities such as surveying, staking out, and various computations are already fully operational. This advancement in GNSS technology represents a notable step forward in the field of geodesy and cartography, even as further refinements and enhancements to the software continue.

Fig. 1. GNSS Nivel Point 3 (photo by author)

- Laser scanner Faro Orbis

Hybrid measurements combining static and mobile scanning modes are made possible by the FARO Orbis laser scanner. This device continues the legacy of the scanner series originally developed by the now-defunct company GeoSLAM, which was acquired by FARO in 2022. Modern scanners are known for their high accuracy and advanced functionality, making them indispensable tools for geodetic applications. One of these scanners offers technical precision up to 5 millimeters, which may be unnecessarily high for certain tasks related to the Digital Technical Map (DTM), but remains highly practical for specialized measurement assignments. The Orbis scanner is capable of mapping areas within a radius of up to 120 meters from the scanning head and includes a built-in 360-degree panoramic camera that enables point cloud coloring, thereby improving data visualization [12]. Additionally, the scanner allows for real-time on-site verification of preliminary data through a connected mobile phone, enhancing both efficiency and measurement reliability.

Fig. 2. FARO Focus (photo by author)

- Total station Sokkia iX1203

The Japanese brand Sokkia, active in the surveying equipment market for more than a century, has been part of the Topcon corporation since 2008. Despite this integration, Sokkia has maintained its brand identity and continues to pursue innovation, particularly in the development of total stations, including robotic models. Its latest product line, the iX 1200 series, demonstrates its technical excellence, with the final digit in the model number indicating angular accuracy. For the purposes of this scientific paper, the Sokkia iX1203 total station was used. This instrument provides an angular accuracy of 3 arcseconds and features a highly precise electronic distance measurement unit with a precision of 1 + 2 ppm [13].

Fig. 3. Sokkia iX1203 (photo by author)

Identical points were first surveyed using a total station and then subsequently measured with GNSS at different time intervals and across various RTK correction networks. Based on findings presented in the article [19] it was possible to exclude the influence of internet service providers. Therefore, this factor is not addressed further in this study.

As already mentioned, the chosen survey site offers optimal conditions, primarily due to the absence of nearby fixed reference stations, with the closest being located more than 18 kilometers away. This is illustrated in the accompanying figure. In addition to the measurement itself, this study also focuses on evaluating the influence of individual RTK correction service providers. The corrections used in the research are sourced from CZEPOS, TopNET, Geoorbit, and VRSnow. The nearest reference station is operated by Geoorbit and is situated approximately 18 kilometers from the Church of Saint Catherine in the village of Jeníkov.

Fig. 4. sketch of the distribution of pernametric stations for RTK corrections (map by author)

In the Czech Republic, there are several correction networks and companies dedicated to providing RTK correction services. These providers are generally categorized into two groups: public and private entities. The public provider is CZEPOS, operated by the Czech Office for Surveying, Mapping and Cadastre, while private sector services include TopNET, Geoorbit, and VRSnow. Each provider employs its own infrastructure and reference stations, which may influence the consistency and reliability of GNSS measurements depending on regional availability and network geometry. Reference system ETRS89 is maintained on all above-mentioned networks through the monitoring of reference stations in the Czech Republic [18].

CZEPOS

The public RTK correction service provider in the Czech Republic is the Czech Office for Surveying, Mapping and Cadastre (ČÚZK), which owns and operates the national network. This network offers multiple mountpoints, of which the following three were primarily used in this study:

VRS – a network-based solution that generates virtual reference stations using data from surrounding fixed stations

MAX – corrections from a main permanent station, adjusted based on input from neighboring stations

Nearest – corrections derived from the closest physical permanent reference station

The state-operated network infrastructure consists of reference stations manufactured by Leica, Trimble, and Topcon. This diversity in hardware may occasionally lead to challenges in data exchange between stations. The system uses Leica Spider processing software for correction distribution and network management [14].

Fig. 5. CZEPOS pernament stations (edited by https://czepos.cuzk.cz/)

TopNET

The private correction service provider is GB-Geodezie, spol. s r.o., which owns and operates its own network. The company offers several mountpoints, of which the following were primarily used in this study:

VRS – a network-based solution generating virtual reference stations from surrounding data

Nearest – corrections provided by the closest physical permanent reference station

This private network is built using permanent stations manufactured by Topcon. Correction data is processed and distributed using the TopNET software platform [15].

Fig. 6. TopNET pernament stations (from http://topnet.gb-geodezie.cz/topnet/)

GEOORBIT

The private RTK correction service provider Geoobchod s.r.o. owns and operates its own correction network. While the company offers a variety of mountpoints, the following were primarily used in this study:

VRS – a network-based solution that generates virtual reference stations from surrounding data

MAX – corrections from the main permanent station adjusted by input from adjacent stations

Nearest – corrections from the closest physical permanent reference station

This private network is built on permanent stations manufactured by CHC Navigation. Recently, the network adopted the Leica Spider processing software for correction data management and distribution [16].

Fig. 7. GEOORBIT pernament stations (from https://www.geoorbit.cz/)

VRSnow

The private correction service provider Trimble Inc. owns and operates its own RTK correction network. While the company manages a variety of mountpoints globally, the only mountpoint used in this study is VRS—a network-based solution that generates virtual reference stations from surrounding fixed stations. The VRSnow service does not offer other mountpoint types. This private network is constructed using permanent reference stations manufactured by Trimble. Correction data is processed and distributed through the Trimble Pivot software platform [17].

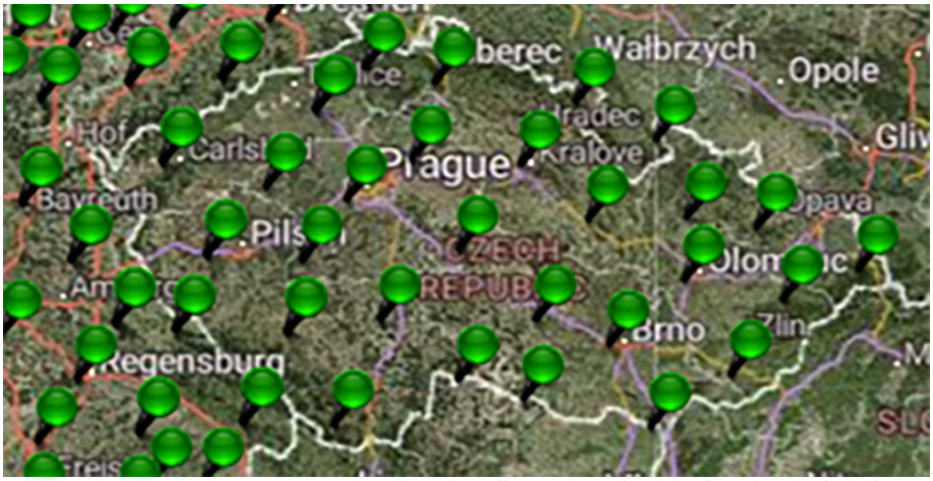

Fig. 8. VRSnow pernament stations (from https://geotronics.cz/produkty/gnss-korekce/o-siti/)

The measurement campaign comprises 25 entries corresponding to distinct identical points. A total of 24 points were selected and precisely surveyed using a total station, including:

8 points along a masonry wall

14 points located at the corners of the church building

2 geodetic control points from the national reference network

All points will be measured without tilt, or with zero tilt, to ensure consistency. GNSS measurements will be conducted using RTK corrections, specifically employing three configurations: correction from the nearest physical reference station, a virtual reference station, and a version utilizing iMax corrections. The survey pole height will remain constant throughout all measurements, set to exactly 2 meters.

The identical reference points were surveyed using a total station and oriented with respect to the geodetic control network managed by the Czech Office for Surveying, Mapping and Cadastre (ČÚZK). Specifically, the points used include No. 922052820, 922052821, 922052822, and 922052590. Points 922052821 and 922052822 were repeatedly measured with GNSS technology during testing, serving as control measurements that allow the evaluation of GNSS accuracy against standardized geodetic reference points. This methodology ensures high data fidelity and provides a reliable basis for assessing measurement quality under various conditions.

GNSS measurements were conducted on March 23, 2025, during both morning and afternoon sessions. The total station measurements took place on February 13, 2025, while control measurements with the mobile laser scanner were carried out on June 10, 2025. The tables below present a comparison between GNSS measurements made with tilt applied and the same points measured using a total station.

Fig. 9. Distribution of test points (map from author)

Firstly, the geocentric coordinates in ETRS89 system were transformed to local reference systems S-JTSK/Bpv using transformation in the firmware of each GNSS receiver. The following formulas were selected for subsequent calculations. Classical statistical approaches were used to determine coordinate deviations

$$\Delta X = x_2 - x_1,$$
$$\Delta Y = y_2 - y_1,$$
$$\Delta Z = z_2 - z_1,$$

In the following calculations, the variables x_2, y_2, and z_2 represent the coordinates obtained from the reference (baseline) measurement, while x_1, y_1, and z_1 denote the coordinates derived from the tested measurement. The xi and yi coordinates represent planar coordinates in the S-JTSK reference system and zi is height in the Bpv reference system. This distinction enables the analysis of positional deviations between the tested data and the standard, allowing for a quantitative evaluation of measurement accuracy:

$$RMS = \sqrt{\frac{1}{n}\sum_{i=1}^{n} h_i^2}$$

Subsequently, the RMS (Root Mean Square) value was calculated according to the following approach: Where:

- h_1, h_2, ..., h_n represent the individual deviation values (one from ΔX, ΔY or ΔZ)
- n is the total number of values considered

This calculation provides a standardized statistical measure of the dispersion of deviations from the reference measurement, allowing for quantitative comparison of accuracy across the tested technologies.

3. Results

Below is a comparison of measurements conducted during the morning and afternoon sessions. This assessment aims to evaluate potential temporal variations in GNSS accuracy under consistent environmental and methodological conditions. By comparing identical points surveyed at different times of day, the study explores the influence of factors such as satellite geometry, atmospheric conditions, and network performance on positional deviations. Based on previous studies [20] we expect higher ionospheric influence in morning session.

Tab. 1a. Morning measurement with best results

point	RTK network	TILT	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]
101	CZEPOS MAX	5°	0.0	-0.2	-0.1	0.13
102	CZEPOS VRS	27°	2.9	-3.4	-0.7	2.61
103	CZEPOS VRS	47°	3.4	-0.6	-0.9	2.06
104	CZEPOS NEAREST	41°	1.9	2.8	-1.6	2.16
105	GEOORBIT VRS	8°	2.6	-0.3	1.0	1.62
106	CZEPOS NEAREST	50°	2.9	-1.7	-4.6	3.29
107	GEOORBIT NEAREST	9°	1.9	-0.4	-1.6	1.45
108	TOPNET NEAREST	35°	4.4	-5.9	1.2	4.31
109	GEOORBIT VRS	1°	2.1	1.6	-0.4	1.54
110	GEOORBIT VRS	2°	2.7	-0.4	-1.4	1.77

Tab. 2b. Morning measurement with best results

point	RTK network	TILT	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]
111	GEOORBIT VRS	35°	2.3	1.2	0.0	1.50
112	GEOORBIT VRS	31°	3.2	1.8	-0.1	2.12
113	GEOORBIT VRS	45°	3.2	-0.6	1.0	1.97
114	TOPNET NEAREST	32°	1.6	1.6	2.9	2.12
1	CZEPOS NEAREST	0°	2.2	-0.4	-0.2	1.30
2	GEOORBIT VRS	39°	0.5	0.6	1.0	0.73
3	TOPNET NEAREST	0°	3.3	0.6	1.5	2.12
4	CZEPOS NEAREST	31°	2.5	-4.4	1.0	2.98
5	CZEPOS VRS	0°	4.3	-3.0	-1.3	3.12
6	CZEPOS VRS	41°	1.9	-2.7	-0.5	1.93
7	CZEPOS NEAREST	71°	1.0	-0.5	-1.7	1.17
8	CZEPOS VRS	23°	0.7	-0.4	-1.3	0.88
922052821	GEOORBIT VRS	1	1.7	-0.1	-0.6	1.04
922052822	CZEPOS NEAREST	1°	-1.2	-1.7	0.4	1.22
Level point	VRSnow	57°	---	---	0.2	0.2

Tab. 3. Afternoon measurement with best results

point	RTK network	TILT	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]
101	VRSnow	42°	0.7	-0.6	-0.3	0.56
102	CZEPOS NEAREST	3°	0.7	-2.0	1.4	1.47
103	VRSnow	57°	2.3	3.2	2.8	2.79
104	CZEPOS NEAREST	32°	2.3	-0.6	0.5	1.40
105	TOPNET NEAREST	32°	1.2	1.3	4.6	2.85
106	CZEPOS VRS	34°	5.8	-0.5	-0.7	3.39
107	GEOORBIT VRS	7°	2.9	0.9	2.3	2.20
108	TOPNET NEAREST	40°	3.3	-5.9	5.8	5.14
109	GEOORBIT iMAX	43°	3.4	-0.5	-3.3	2.75
110	CZEPOS VRS	36°	3.3	0.9	-1.7	2.21
111	CZEPOS NEAREST	39°	1.7	-0.9	2.0	1.60
112	CZEPOS MAX	34°	1.3	1.9	0.4	1.35
113	CZEPOS VRS	46°	1.8	-0.6	-4.1	2.61
114	VRSnow	0°	2.5	1.3	1.0	1.73
114	VRSnow	39°	0.1	-2.6	1.5	1.73
1	CZEPOS NEAREST	16°	0.6	-1.8	-0.4	1.12
2	GEOORBIT VRS	27°	-1.5	0.3	-0.2	0.89
3	CZEPOS MAX	0°	1.7	-0.3	-2.1	1.57
4	VRSnow	0°	4.9	-1.6	1.8	3.15
5	CZEPOS VRS	0°	4.8	-1.6	1.0	2.98
6	VRSnow	1°	3.7	-1.8	2.8	2.87
7	CZEPOS MAX	43°	0.5	-1.2	-0.8	0.88
8	GEOORBIT VRS	0°	1.2	-1.4	-0.1	1.07
922052821	VRSnow	38°	1.7	0.4	1.8	1.45
922052822	GEOORBIT VRS	0°	2.4	-2.6	0.1	2.04
Level point	VRSnow	90°	---	---	-0.1	0.1

The following table isolates extreme positional deviations identified during the GNSS measurements in comparison with the total station reference points. These outliers may indicate anomalies caused by multipath effects, atmospheric disturbances, or temporary network inconsistencies, and they are excluded from the final accuracy analysis to ensure the reliability of statistical evaluations.

Tab. 4a. The biggest differences between the measurements

point	RTK network	TILT	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]	morning/afternoon
101	GEOORBIT NEAREST	10°	3.7	-4.1	-6.7	5.01	morning
102	CZEPOS NEAREST	3°	7.9	6.6	6.9	7.15	Morning
103	CZEPOS MAX	11°	28.4	-10.0	49.0	33.20	Morning
104	TOPNET NEAREST	10°	1.1	-12.0	11.1	9.46	Morning
105	TOPNET NEAREST	23°	1.5	2.8	14.7	8.68	Morning
106	VRSnow	7°	16.2	-12.2	-13.3	14.00	Afternoon
107	TOPNET NEAREST	39°	3.0	4.6	11.4	7.31	Afternoon
108	CZEPOS VRS	3°	44.7	-20.2	-50.4	40.60	Afternoon
109	GEOORBIT iMAX	2°	16.4	-2.7	-13.9	12.51	Morning
110	GEOORBIT iMAX	2°	17.1	-6.2	-15.1	13.65	Morning
111	GEOORBIT iMAX	8°	16.6	-3.4	-13.2	12.40	Morning
112	GEOORBIT NEAREST	10°	7.1	-7.0	-2.8	5.98	Morning

Tab. 5b. The biggest differences between the measurements

point	RTK network	TILT	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]	morning/afternoon
113	CZEPOS VRS	4°	11.7	28.3	-26.8	23.49	Morning
114	TOPNET NEAREST	41°	3.7	-6.3	8.0	6.26	Afternoon
1	GEOORBIT NEAREST	37°	3.7	-3.9	-4.8	4.16	Afternoon
2	GEOORBIT NEAREST	41°	1.7	2.4	-5.2	3.45	Afternoon
3	CZEPOS VRS	37°	7.3	-4.3	-5.2	5.74	Afternoon
4	GEOORBIT NEAREST	30°	5.0	-7.0	-6.5	6.22	Afternoon
5	GEOORBIT NEAREST	34°	6.2	-7.1	-5.6	6.33	Afternoon
6	VRSnow	0°	7.6	-6.9	0.6	5.94	Morning
7	GEOORBIT NEAREST	0°	5.8	-4.4	-4.0	4.80	Afternoon
8	GEOORBIT NEAREST	34°	3.6	-1.5	-5.9	4.08	Afternoon
922052821	GEOORBIT NEAREST	0°	2.7	-4.5	-5.1	4.22	Afternoon
922052822	GEOORBIT iMAX	0°	-2.8	-8.5	-2.9	5.43	Afternoon
Level point	GEOORBIT iMAX	57°	---	---	-15.3	15.3	Morning

The final table presents an overview of which RTK correction network yielded the highest number of extreme deviation values based on the available measurements.

Tab. 6. Correction mountpoints with worst results

RTK network	freqency
CZEPOS VRS	3
CZEPOS NEAREST	1
CZEPOS MAX	1
TOPNET VRS	0
TOPNET NEAREST	4
VRSnow	2
GEOORBIT VRS	0
GEOORBIT NEAREST	9
GEOORBIT iMAX	4

The following section presents a comparison between points extracted from the point cloud and those GNSS measurements that exhibited the lowest positional deviations. Since the point cloud was georeferenced using control points previously surveyed with the total station, any direct comparison with the total station itself would not be meaningful. Instead, the analysis focuses on validating the internal consistency and accuracy of the mobile laser scanning data in relation to independently acquired GNSS measurements.

Fig. 10. Screenshot of point clouds from FARO orbis (from 3Dsurvey, author)

Tab. 7. Comparions between GNSS and points from point clouds

point	dY [cm]	dX [cm]	dZ [cm]	RMS [cm]
101	2.8	1.7	1.0	1.98
102	2.7	-0.3	1.2	1.71
103	10.6	-0.5	1.3	6.17
104	-0.1	-9.6	4.8	6.20
105	2.9	2.3	-1.4	2.28
106	3.3	0.1	-4.6	3.27
107	2.0	-0.3	-2.3	1.77
108	3.0	-5.5	0.5	3.63
109	0.2	1.7	-0.9	1.12
110	1.8	-3.6	-1.4	2.46
111	3.7	0.7	0.9	2.24
112	0.8	3.3	1.0	2.04
113	6.2	0.1	3.5	4.11
114	4.0	3.2	3.1	3.46
1	3.4	0.4	0.0	1.98
2	2.6	0.3	-2.7	2.17
4	1.7	-0.8	3.0	2.04
5	7.4	2.1	4.8	5.23
922052821	2.3	-2.1	0.1	1.80
Level point	---	---	-1.2	1.20

5. Conclusions

One of the initial hypotheses was that GNSS measurements taken in the morning would be more affected by external influences namely ionosphere than those taken in the afternoon. However, this assumption was not confirmed during the timeframe of this particular experiment. On the other hand, the second hypothesis—concerning the impact of the type of RTK correction network (VRS vs. NEAREST)—showed stronger evidence. The highest number of inaccurate measurements occurred when using the NEAREST correction type, in which the reference station vector ranged between 18 and 25 kilometers from the measured points.

Out of the 25 measured points, nine exceeded the threshold values for positional accuracy: 14 cm in horizontal deviation and 12 cm in elevation. These extreme deviations primarily occurred in elevation, while horizontal discrepancies were most commonly observed along the Y-axis. The points with the greatest errors were numbers 103 (RMS: 33 cm), 108 (RMS: 41 cm), 113 (RMS: 23 cm), and 106 (RMS: 14 cm). These points correspond to corner locations on the surveyed object, where satellite signal obstruction is likely to occur due to surrounding structures.

An additional finding concerned the reliability of elevation measurements at a leveling benchmark located on the church building. Due to extremely high position of benchmark above ground the GNSS measurements at this point were taken with a steep tilt, resulting in an unexpectedly large deviation of 15 cm at a tilt angle of 57°.

No significant complications arose during data collection. In some cases, extended waiting times for RTK FIX initialization were observed, but these delays did not substantially impact the overall measurement workflow.

This study does not mark the conclusion of the research effort. Several open questions remain, such as whether the observed measurement deviations are seasonally dependent, whether varying the pole height has a measurable effect, and whether local correction methods, such as base–rover setups with UHF transmission, offer improved accuracy and reliability. There is also interest in exploring the potential benefits of real-time PPP (Precise Point Positioning). These topics are expected to be addressed in future measurement campaigns.

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