Assessment of the Accuracy of Methods of Mobile Laser Scanning

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Abstract

This paper describes a mobile scanning system Lynx Mobile MAPER and its use in the acquisition of survey documentation - 3D model of the yet unused operationally D11. Based on different configurations of ground control points (their mutual separation) and determining their position with classical geodetic methods, there were evaluated the achieved accuracy of positioning control points checked using mobile laser scanning technology.

Key words: mobile scanning, compare the accuracy of measurement, terrestrial measurement methods, ground control point

1 INTRODUCTION

Terrestrial and mainly mobile laser scanning is a relatively young technique. Only in the last few years it is getting to the forefront thanks to developing of advanced technologies and is thus used in many human activities such as the mapping of line constructions and their surroundings, change detections and generation of accurate digital models to form a universal 3D background for all geo-located applications.

2 THEORY AND EQUIPMENT

The basis of the mobile mapping laser system (MMLS) consists of two integrated units: the scanning unit with laser scanners and the navigation unit so-called the Position and Orientation System (POS).

2.1 LASER SCANNERS

The used acronym LIDAR (Light Detection And Ranging) suggests the system principle. A pulse laser sends hundreds of thousands of rays in the infrared spectrum per second (100-500 kHz) and records the time of their reflections back. Individual beams are spread in the transverse direction by the rotating mirror with a scan frequency of up to 200Hz around the axis of the scanning head and by moving of the platform (car, boat, rail vehicle) a continuous strip of "point clouds" along the trajectory of driving is scanned (Fig. 1). From each emitted beam can be recorded multiple reflections which depends on surroundings - how the beam passes through the scanned materials (e.g. vegetation). In addition to this information about the number and order of the reflection, for each point it is recorded for example information about the intensity of reflection, scan angle and GPS time as well. Used LIDARs comply with the strictest requirements according to safety Class 1 (eye-safe laser) (Tab. 1).

Figure 1 Surveying vehicle with LYNX M1 and POS LV 520

Table 1 Technical overview of LYNX Lidar M1

LYNX Lidar M1	
Scanner field of view	360°
Laser classification	IEC/CDRH Class 1 eye-safe
Scan frequency	500 kHz programmable
Laser measurement rate	250 Hz, programmable
Range precision	+/-8 mm, 1 sigma
Measurement per laser pulse	Up to 4 simultaneous
Maximum range	200 m, 20% reflectivity

2.2 POSITION AND ORIENTATION SYSTEM

POS provides precise spatial trajectory of the moving platform in real time (SBET - Smoothed Best Estimate Trajectory) and then allows the exact localization of each scanned point. One part of POS consists of gyroscopes and accelerometers system (IMU-Inertial Measurement Unit), which records tilts and accelerations, and Distance Measurement Instrument (DMI), whereas the second navigation segment provides positioning based on GNSS (Global Navigation Satellite System). This system consists of antennas mounted on a platform and a GNSS device located on a reference point close to the scan site. The whole system is further supplemented by a set of two or more digital cameras which acquire image recordings of the measured location simultaneously with scanning (Tab. 2).

Table 2 Performance summary of Applanix POS LV 520

POS LV 520	With GNSS*			GNSS Outage, 60 seconds*		
	PP	IARTK	DGPS	PP	IARTK	DGPS
X,Y Position (m)	0.020	0.035	0.300	0.100	0.300	0.420
Z Position (m)	0.050	0.050	0.500	0.070	0.100	0.530
Roll and Pitch (°)	0.005	0.008	0.008	0.005	0.008	0.008
True Heading (°)	0.015	0.020	0.020	0.015	0.020	0.020
* All accuracy values given as RMS. Assumes typical road vehicle dynamics for initialization.						

2.3 PREPARATORY WORK

On the new still unopened part of highway D11 in the open countryside, there was signated geodetic point field. Points are spread out around the edges and in the middle of the road in the direction of Prague in intervals approximately 10, 20 and 50 m. There were sprayed two white opposite circle segments of radius 0.3 m on the road surface through a metal template. Touching of the tops of the segments determines the surveyed control points (Fig. 2).

Figure 2 Road surface with geodetic point field

2.4 MEASUREMENT

Surveying with Lynx Mobile Mapper was done on 2nd July 2013. Measurements were carried out at speeds of 20, 30, 50 and 70 km / h always at both edges and in the middle of the road and at a scan speed of 500 and 250 kHz.

As a reference GNSS base station was used ProMark 500 based on the trigonometric point no 000916250300.

Note: The manufacturer guarantees absolute accuracy of the point cloud up to 5 cm under the following conditions:

- Min. number of satellites in solution 6
- Max. PDOP 3
- Max. distance to GNSS base station 20 000 m

Conditions during the measurement:

- Min. number of satellites in solution 8
- Max. PDOP 1.67
- Max. distance to GNSS base station 1 840 m

2.5 DATA PROCESSING

SBET (Smoothed Best Estimate Trajectory)

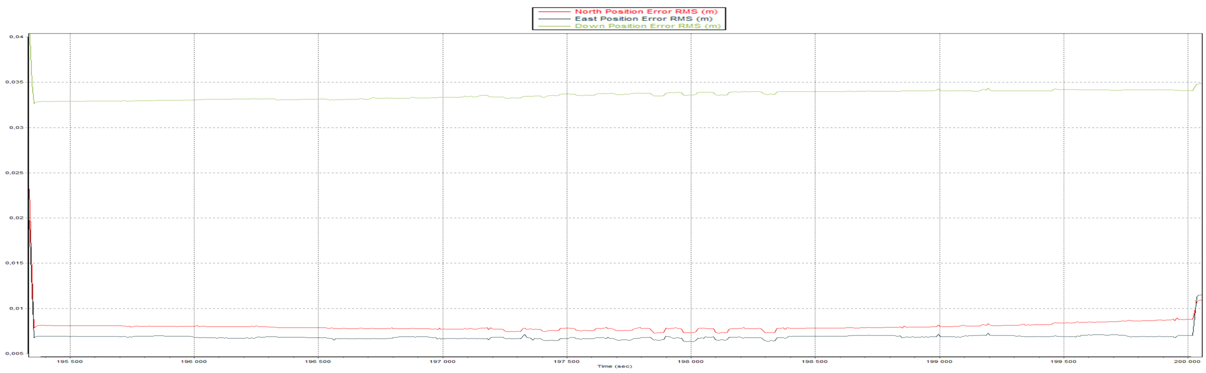
In POSPac MMS (Applanix software module) was processed POS and GNSS base data to create SBET in WGS-84 and ellipsoidal heights.

Point Cloud Generating

After importing the SBET and raw data from Lidars into DashMap processing software the point cloud were generated for each strip.

Matching and controlling

During processing in TerraScan and TerraMatch (TerraSolid software systems) each overlapping laser strip from different lane (2 lanes) was compared and checked with control points each second point on the edge of the road and fixed points (i.e. on bridge or pylons). After finding the best fit it was adjusted and matched.



Graph 1 Positions Errors RMS (m)

Final data was transformed to local coordinate system using software Krovak 1.0.1.1 approved for transformation between ETRS89 and S-JTSK.

From the final point cloud, coordinates of control points were evaluated and then compared with the coordinates acquired by classical methods.

Technology comparing the accuracy is based on comparison of coordinates of checked points determined by the mobile scanning with the coordinates determined independently by the terrestrial method – trigonometric determination of coordinates using the total station Leica TCA 2003 in relation to the state fundamental horizontal control, GNSS technology for determining the coordinates Y, X, Z and geometric leveling for determining the heights of points.

In terms of the used coordinate datum, we use the S-JTSK, (Datum of Uniform Trigonometric Cadastral Network), in case of GNSS technology, we use the S-JTSK (CR 2005). Regarding the elevation determination, we speak about the Baltic Vertical Datum.

The controlled geodetic point field is situated in two lanes of motorway in the direction from Hradec Králové to Praha. Points are indicated on the outside edge of the outer extreme lane and on the inner edge of the travel lane in remoteness of 10 meters on 100 m long section, followed by the remoteness of 20 meters on 200 m long section and the remoteness of 50 m on 650 m long section. In addition, all central points of such formed shapes (squares and rectangles) are marked. See Figure 2 for marked points and Figure 3 for the numbering.

The checked motorway section is configured in the x-axis direction, i.e. the x-values change, but the y-values remain virtually unchanged.

2.6 IMPLEMENTATION OF THE GEODETIC BASIS FOR ASSESSING THE ACCURACY OF THE MOBILE SCANNING, DETERMINATION OF GROUND CONTROL POINTS AND CHECKED POINTS

2.6.1. Determining the heights of signalized points

To determine the heights of all signalized points, the very precise leveling was made between the point 3090 (altitude of 235.066 m) and the point KH-0055-9 (altitude of 230.875 m). The checked points themselves were determined by precise leveling in relation to the points determined by very precise leveling.

To evaluate the determination of heights by GNSS (comparative measurements of the VUGTK), the height of the checked points were determined twice, independently, with the necessary time delay and two GNSS apparatuses.

2.6.2. Positioning of signalized points

Positioning of all checked signalized points was made by two technologies. The terrestrial determination in relation to the state point field (S-JTSK) and the GNSS technology.

To determine the positions of all signalized checked points, sets of directions and corresponding lengths of sight lines were observed from stations 4001 and 4002 to the S-JTSK state network points using Leica TCA 2003. All controlled signalized points were also determined by GNSS technology (Křovák 2013, quasigeoid model CR 2005). The measurement was performed twice, independently and with necessary time delay.

Figure 3 Road surface with geodetic point field

3 ASSESSMENT OF ACCURACY OF THE MOBILE SCANNING

Evaluation of accuracy of the mobile scanning itself was performed by assessment of coordinate differences of checked signalized points as follows:

- By comparing to the results obtained from the terrestrial measurements using TCA 2003 and utilizing 50 % of all control points (every other one) in the evaluation of mobile scanning.
- By comparing to the results obtained from the terrestrial measurements using TCA 2003 and utilizing 100 % of all control points in the evaluation of mobile scanning.
- By comparing to the results obtained by the GNSS technology utilizing 50 % of all control points (every other one) in the evaluation of mobile scanning.
- By comparing to the results obtained by the use GNSS technology utilizing 100 % of all control points in the evaluation of mobile scanning.

Evaluation of accuracy was also done at the division of the entire highway section in three subsections with signaling the points by 10 m, 20 m and 50 m.

Based on an evaluation of each considered variant, we can say:

- Accuracy of the mobile scanning in relation to the results of terrestrial determination was reached, in case of control point distances up to 20 m, or 40 m (X, Y), the same and can be characterized by the standard coordinate deviation $\sigma_{x,y}$ = 10 mm. In case of control point distances of 50 m and 100 m, the standard coordinate deviation $\sigma_{x,y}$ was 18 mm. Greater inaccuracy was observed in Y coordinate.

It results in recommendations for signaling the control points in relation to the accuracy of the methodology:

- Accuracy of the mobile scanning in relation to the results of the terrestrial determination within the elevation meaning can be characterized for the above mentioned variants by identical standard deviation σ_z = 12 mm.
- Accuracy of the mobile scanning in relation to the results of the GNSS technology measurement verification, i.e. virtually for the same technology, was reached with the remoteness of the control points of 10-20 m, or 40-100 m (X, Y) practically the same and can be characterized by the standard coordinate deviation $\sigma_{x,y}$ = 12 mm. Regarding the heights, the above given variants can be characterized by the same standard deviation
  σ_z = 7 mm.
- When comparing the accuracy of the coordinates of the checked points determined by terrestrial technology and GNSS technology, the accuracy for the remoteness of control points up to 20m, or 40 m (X, Y) is practically the same and can be characterized by the standard coordinate deviation $\sigma_{x,y}$ = 10 mm. For the remoteness of the control points 50 m and 100 m, the achieved standard coordinate deviation was $\sigma_{x,y}$ = 18 mm, with greater inaccuracy in Y coordinate.

It results in recommendations for signaling the control points in relation to the accuracy of the methodology.

When comparing the accuracy of the coordinates of the checked points determined by the terrestrial technology and by GNSS technology, the standard coordinate deviation $\sigma_{x,y}$ = 17 mm was obtained. Regarding the heights, the above stated variants can be characterized with the same standard deviation σ_z = 12 mm.