Mobile Laser Scanning Technology in Road Depression Measurements

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ABSTRACT

This research paper describes applicability of mobile laser scanning in road depression measurements. The research was commissioned by the Finnish Transport Agency and performed by Destia Ltd. The applicability was researched by examining accuracy, precision, and repeatability of the mobile laser scanning technology. Two about one kilometre long road segments near Porvoo, Finland, were chosen for test sites. Both of them were measured twice with two different laser scanning devices, Trimble MX8 and Riegl VMX-450. Then through a modelling process, 3D surface models were produced from the targets. Also a reference model was made with a levelling instrument from both of the road segments. By comparing these models, the accuracy of the model's z-coordinates and longitudinal slopes was examined. Results show that mobile laser scanning has clear promise in road depression measurements. For example, both of the devices used can achieve an average z-accuracy close to zero with about 10 mm dis-persion. This is be adequate enough in some cases. It is also possible to improve the accuracy by developing the surveying method. Furthermore, repeatability showed promising results. Some further research and definitions have to be made, although the results were very promising already at this point.

This conference paper is a summary of the original report that is accessible in the Finnish transport agency's database [1].

Keywords

Mobile Laser Scanning, Lidar, Point Cloud, Road Depression, 3D-model

1 Introduction

1.1 Background

Road depression measurements are traditionally made with levelling. This method requires a significant amount of manual measurements. Usually the measurements are done while the road is in use. Due to these facts levelling is quite laborious and therefore expensive and it also has work safety issues because of the ongoing traffic. Consequently the Finnish Transport Agency has wanted to look for an alternative for the levelling measurements.

Depression measurements are used to study the conditions and quality of a road surface. The measurements can be done for example right after a new road has been completed and when the roads guarantee expires. They can also be done between these moments for example once every year. If the case is that guarantee is expiring, the information gathered from the depression measurements are then used to define if the contractor has to repair the depressed road or if the purchasing price is decreased. Sometimes it is a question of significant amounts of money. Therefore depression measurement methods have to possess great accuracy and precision.

Mobile laser scanning was selected as a possible alternative due to it's applicability in other types of road surveying. For example, Destia Ltd. has used mobile laser scanning for producing source data models for road design projects. Asphalt rehabilitation design is one of the most useful applications of mobile laser scanning [2]. Depression measurements require a high level of accuracy from the used method. Through these previous projects, there was a clear idea that the accuracy and precision of mobile laser scanning technology could be sufficient enough for depression measurements. Moreover, this technology is safer, more effective and could also provide a more comprehensive picture of the roads condition through 3D-modelling. [2,3]

1.2 Traditional depression measurements

Road depressions can be considered from two perspectives. First is to examine the absolute depression in vertical direction. The other perspective is to research the actual harm and discomfort experienced by the user of the road. [4]

Measurement of the absolute depression can be demonstrated with figure 1. Two different depressions, h_1 and h_2, can be seen in figure 1. Both of these express the difference in absolute heights between road surface at time of completion (t_0) and at some chosen longer period of time. In many cases the time period is the end of the guarantee. h_2 could be acceptable and h1 could be deep enough to have consequences for the contractor. The limits for the magnitude of h1 and h2 are usually defined in the contract documents. [4]

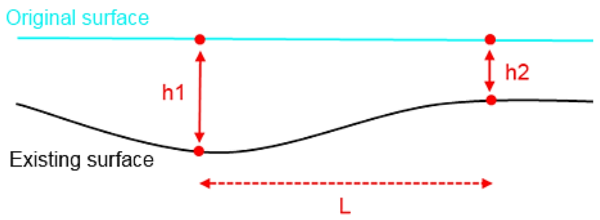


Figure 1. Depression of road surface between original and existing surfaces.

Also the length (L) in figure 1 is a significant factor. For example, if there is some municipal infrastructure like sewers and water pipes located under the road, they might break if the depressions of different magnitude are located too close to each other. [3]

The second perspective to examine road depression is to assess the actual harm and discomfort caused to the road user. That cannot be done just by examining the absolute depressions. If the whole road depresses the same amount, no harm is caused to the driver. When the depression is different in two subsequent spots of the road, some discomfort can appear while driving the road. The shorter the distance between the depressions is, the greater the discomfort is. This can be seen in figure 1.

The same phenomena can be seen also in figure 2. Figure 2 also demonstrates the method used for existing roads when there is no surveyed information from the time of completion. Two longitudinal slopes, between points can be calculated from the information seen in figure 2. Also the angle between them can be calculated. This angle or change in slope describes the driving discomfort caused by the road surface. At this time the depression calculations are done with this simple mathematical method. Mobile laser scanning and 3D-modelling are hoped to evolve the used method.

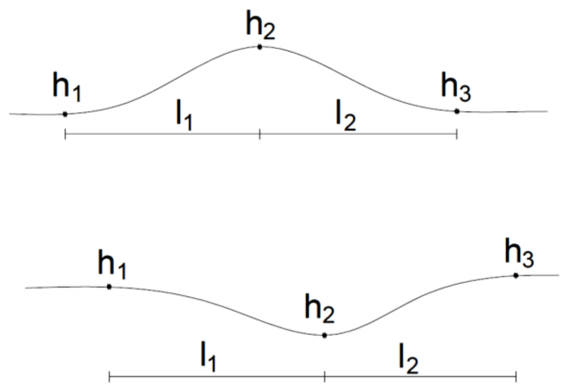


Figure 2. The principle and data points for calculating changes in subsequent longitudinal slopes on road surface [5]

2 Surveying Methods

2.1 Test sites

The roads used as test sites were located near Porvoo, Finland. The accurate locations in Finnish road address system were as follows:

1. Mt 170, road segment 10, distance 400 - 1400 m,
2. Kt 55, road segment 3, 3075 – 3990 m.

The test sites were chosen so that both have visually clear depressions and the road environments differ from each other. Site no. 1 was surrounded by forest so it would be challenging considering GNSS-positioning. Site no. 2 was located mainly in the middle of a flat field except for a sparse forest around the half point. Site no. 2 had also bridge crossing it from above at the start. Generally site no. 2 was less challenging for positioning.

2.2 Reference measurements

A number of reference measurements had to be carried out for the mobile laser scanning and accuracy analysis. All reference measurements were made with levelling. Total station was also tested and considered at first but the results between these two instruments differed and levelling was decided to use. Levelling is the more accurate method by default.

Firstly the levelling was used to produce the reference points which were used to analyse the accuracy of the surface models. Reference points were measured along the paint lines on the edges of the road with 25 m spacing and in clear depression areas with 5 m spacing.

Part of the reference points were used as signal points for the signalized mobile laser scanning method. One point every 200 m on both sides of the road was used as a signal point. Signal points had a paint pattern around them which other reference points did not have.

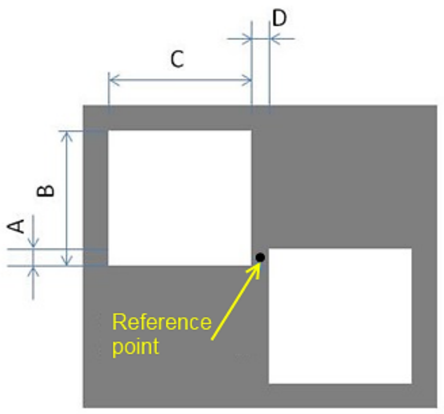
2.3 Mobile laser scanning

2.3.1 Unsignalised and signalised method

Altogether mobile laser scanning was done with three different methods: unsignalised (GNSS-positioned), signalised and RTK-positioned. In this case unsignalised laser scanning means that signal points or RTK-base stations are not used to improve the initial positioning and consequently the final quality of the 3D-models. The positioning was done through GNSS-satellites and therefore there should not be any obstacles more than 15° above the horizon. This is why, for example, surrounding forest makes the positioning more challenging. The advantage of the unsignalized method is that it does not need any surveying measurements to support it. On the other hand, the result might differ tens of centimetres from the actual position of an individual point on the road surface.

Signalised laser scanning is otherwise the same than unsignalised, but signal patterns are painted on the pavement around known points before the scanning. The signal pattern is presented in figure 3. The significance of the white paint pattern is that it can be identified from the point cloud based on intensity. The exact position of the signal patterns were measured with levelling. This information is used to calibrate the point clouds positioning in the data processing phase.

In both cases, after the laser scanning, the data is pre-processed. A trajectory is calculated based on the positioning data and the point cloud is placed to a coordinate system.



A= 1 cm
B= 20 cm
C= 20 cm
D= 1 cm

Figure 3. Paint pattern of the signal points used in data calibration.

2.3.2 Base station method

In the base station method, RTK-base stations were used to improve the initial positioning. RTK-base station is a GNSS-receiver in a known location or fixed point. When the positioning is done through the station, a correction can be calculated afterwards for the measurement data. The final accuracy of the processed data should be as accurate as in the signalized method.

Pre-processing is made with the positioning data acquired from the base stations. The trajectory is calculated and point cloud is set to a coordinate system. The data is very accurate already at this point. The advantage of the base station method is that every point of the trajectory has positioning information from to base stations. Consequently the better one can be chosen.

2.4 Data processing

Data processing of mobile laser data for unsignalised and signalised method is described in detail in reference no. 1. The principle is that the different drive paths are fitted together with the signal patterns, other paint markings and flat surfaces. If paint markings are missing, also other objects such as poles or rails can be used.

Same principles are used for the data acquired by base station method, although calibration with signal patterns is not needed.

After calibration, final products can be produces from the point clouds. They can be for example classified asphalt surface, vectorised break lines from the center of the road and from the edges.

2.5 Precision and accuracy analysis

2.5.1 Accuracy of the z-coordinate

To analyse the accuracy of the surface models z-coordinates, they were compared to levelled reference points. The pavement surfaces classified from the point clouds were triangulated and then the height (z-coordinate) was compared to the reference point's height at the same xy-location. This comparison was made for every produced surface model.

Through these comparisons graphical charts were made to demonstrate the anomalies of the z-coordinates along the roads edges. Also statistical variables such as averages and dispersions were calculated.

2.5.2 Accuracy of longitudinal slopes

For analysis of longitudinal slopes, z-values were red along of the roads center line and edges with 1 m spacing. The reading of the z-values was made by interpolating the height by taking into account nearby points with 15 cm radius. From this data longitudinal slopes were calculated for every surface model individually.

The slopes were calculated with the method demonstrated in figure 4. The first calculation was done the original L = 1 m spacing. Also spacings of L = 2, 5, 10 and 20 m were used. These are the quite commonly used lengths when calculating depressions and ground frost deformations. With the longer spacings, the strings were sled forward so that every data point along the break lines was used.

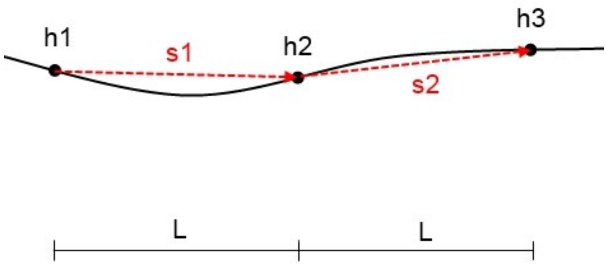


Figure 4. The priciple of calculating and comparing two subsequnt longitudinal slopes on road surface. h = height (m), s = slope (%), L = length/distance (m).

In a situation described in figure 4, longitudinal slope was calculated with equation (1). In the graphical charts the result is presented at the station in the middle of distance L.

$$s_1 = \frac{(h_1 - h_2)}{L} \qquad (1)$$

where s_1 = longitudinal slope [%],
h_1 = height no. 1 [m],
h_2 = height no. 2 [m],
L = distance between h_1 and h_2.

In addition to the longitudinal slopes also the changes of them were calculated. The difference between two subsequent slopes was calculated with equation (2).

$$\Delta s = s_1 - s_2 = \frac{h_1 - h_2}{L} - \frac{h_2 - h_3}{L} \qquad (2)$$

where Δs = change in longitudinal slope [%]
$s_{1\text{-}2}$ = longitudinal slope [%],
$h_{1\text{-}3}$ = height [m],
L = distance between two height points.

The longitudinal slopes were finally compared between different surface models. The comparisons included the following:

1. 1. Signalised vs. 2. signalised
2. 1. unsignalised vs. 2. unsignalised
3. signalised vs. unsignalised
4. 1. base station model vs. 2. base station model

2.5.3 Repeatability

Repeatability was also examined with statistical analysis. The analysis included distributions of height differences between the different models, correlations and Gage-R&R –tests. All of these tests were made with the same height data as the slope accuracy analysis.

The Gage-R&R analysis is used as a quality factor in Finnish Transport Agency's other road quality measurements. Therefore it was also rational to try it to mobile laser scanning technology. Gage-R&R is also part of the "Six Sigma" –quality management method. [5]

The result of Gage-R&R is a percent figure. If the result is <10 %, the repeatability is considered to be excellent. <20 % is a good result, <30 is satisfactory and >30 % is a bad result. Measurements with result of >30 % are usually disqualified which is also the policy in the Finnish Transport Agency. [5]

3 Results

Due to the conciseness of this conference paper, only test site no. 2 is presented on the result-section. Test site no. 2 was considered to be more suitable for making conclusions.

3.1 Reference measurements

The final reference points were surveyed with levelling. Total station was also tried out, but it turned out that there was a systematic difference between these two surveying methods. The z-coordinates from the total station differed 4,7 mm on average from the z-values measured with levelling. The same phenomenon has been noticed also on other occasions. No unambiguous reason was found for this difference. One reason was considered to be the shape of the surveying instrument's tip. The tip of the pole in total station is sharp and the tip of the levelling instrument is flat.

The used total station was Leica TCA 2003. The used levelling instrument for the final points was Leica NDA03. Fixed reference points of The National Land survey of Finland were used in the levelling. The starting point was no. 87M1102C. The closing point was no. 513407.

The layout of the reference points is presented in figure 5. Signal points were painted on the shoulder of the road with 200 m spacing so that they were located on stations 1 m, 199 m, 399 m, 598 m, 796 m and 995 m. The reference points excluding signals were measured along the paint line on the edges of the road. The spacing was 25 m or 5 m in areas with clear depressions .

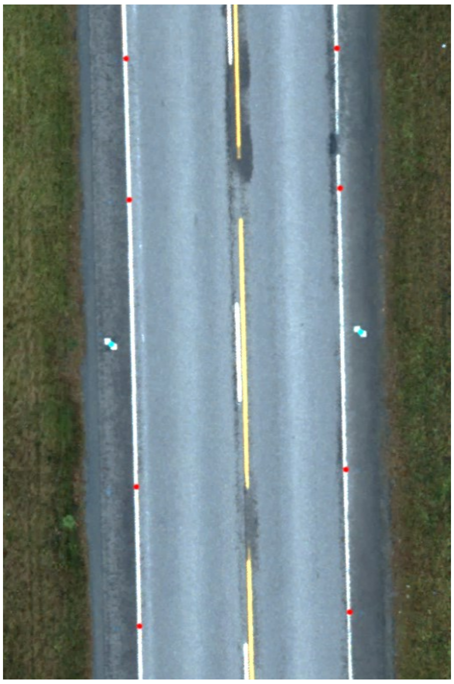


Figure 5. Layout of the reference points (red) and signal paint patterns (white with blue dot).

3.2 Mobile laser scanning

The laser scanning for the signalised and unsignalised method was done Geotrim Ltd. with their Trimble MX8 –device. The first surveying was done during the morning of 20.5.2014, 10-12 o'clock. The second round was performed during the afternoon 13-15 o'clock. The measurement of the test site itself did not take 2 h, but there were other sites included during the same trip. The surveying speed was about 50 km/h. During the surveying Jussi Leinonen from Destia Ltd. was the driver and Anna Klemets from Geotrim Ltd. was the operator of the laser scanning equipment.

The laser scanning with base stations was done by Nordic Scan Center Ltd. Tauno Suominen was the acting project manager. The surveying was done with their Riegl VMX-450 mobile mapping system on 15.10.2014 9-15 o'clock. Base stations were set by Destia Ltd. Used devices were Voif A30 GPS-base stations. Measurements were done once during the morning and once during the afternoon as on the signalised and unsignalised method. During the base station measurements the amount and geometry of GPS-satellites were significantly better during the morning.

3.3 Data processing

Six (6) different 3D-surface models of the test site were produced as a result of data processing phase. Two from unsignalised data, two from signalised data and two from base station data.

The data calibration and forming of the surface model was performed with Terra Scan, Terra Match, Terra Photo and Terra Modeller programs. Although the surveying data was the same for the signalised and unsignalised method, the models could be separated from each other during the data calibration. This was done by forming the unsignalised surface model from the point cloud before applying the signal pattern calibration.

The data from the base station measurements was calibrated with Ri-Process and Ri-Precision programs. Surface models were made with Terra Scan.

3.4 Precision and accuracy analysis

3.4.1 Height accuracy of the models

The height comparisons between the surface models and reference points were done with Terra Scan. The tool triangulates the surface and then compares the height of the surface to the reference point at the same (x,y)-location. The results can be seen on figures 6-8 and tables 1-3. As expected, results were quite similar between the signalized and base station method and the unsignalized method was clearly the most inaccurate.

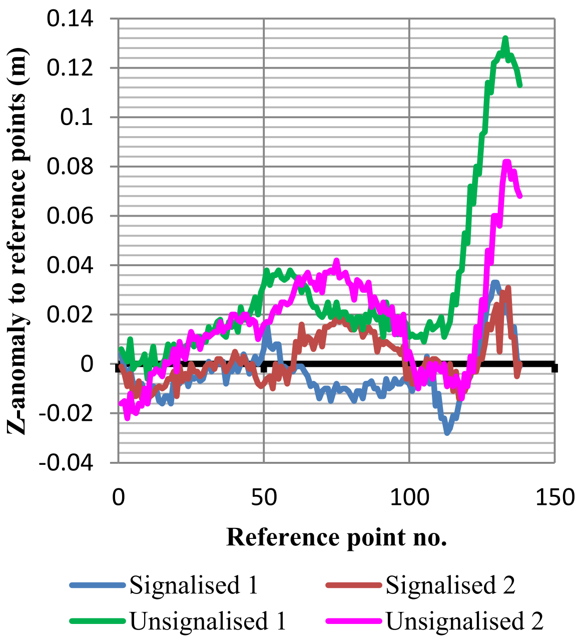


Figure 6. Height anomalies of the models produces with the signalized and unsignalized method (Trimble MX-8)

Table 1. Statistical variables of the anomalies of the models produces with the unsignalised method (Trimble MX-8)

	Kt 55	
	Measurement 1	*Measurement 2*
Avg. Dz (m)	0.029	0.017
Min dz (m)	-0.006	-0.022
Max dz (m)	0.132	0.082
Dispersion (m)	0.033	0.022

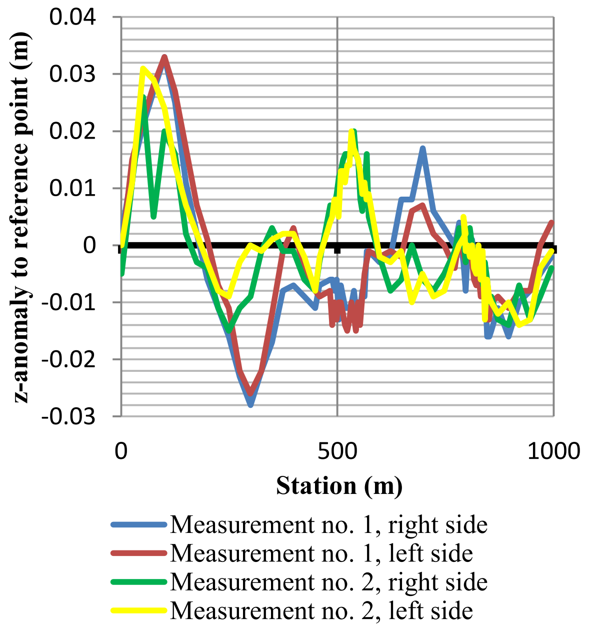


Figure 7. Height anomalies of the models produces with the signalized method (Trimble MX-8)

Table 2. Statistical variables of the anomalies of the models produces with the signalised method (Trimble MX-8)

	Kt 55	
	Measurement 1	*Measurement 2*
Avg. Dz (m)	-0.004	0.002
Min dz (m)	-0.028	-0.014
Max dz (m)	0.033	0.031
Dispersion (m)	0.011	0.010

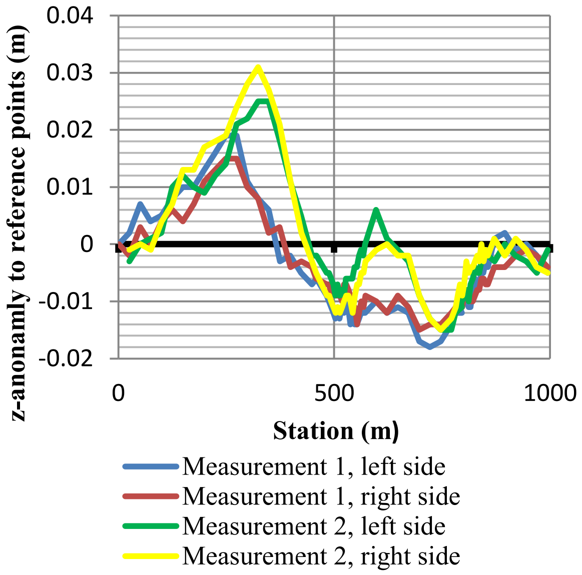


Figure 8. Height anomalies of the models produces with the base station method (Riegl VMX-450)

Table 3. Statistical variables of the anomalies of the models produces with the base station method (Riegl VMX-450)

	Kt 55	
	Measurement 1	*Measurement 2*
Avg. Dz (m)	-0.005	-0.001
Min dz (m)	-0.018	-0.015
Max dz (m)	0.019	0.031
Dispersion (m)	0.008	0.010

3.4.2 Longitudinal slope accuracy of the models

The analysis of the longitudinal slopes is summarised in figures 9-13. The results cover measurements from Kt 55 centreline. These results can be used to demonstrate the phenomena that appeared in all of the results. The results are presented in detail in the original report. Figures 9 and 11 show the actual differences in subsequent slopes on the road surface. These figures would also be the ones to be used for estimating the actual discomfort caused to the driver. The used length for L has a clear effect on the results. Depressions seem to stand out better with the longer L. Figures 10 and 12 show the difference between 1st and 2nd measurement. Differences are smaller with longer L.

Figure 13 represents the difference between signalised and base station method. The differences are generally quite marginal and also here the length of the L has a major effect on the differences between the methods.

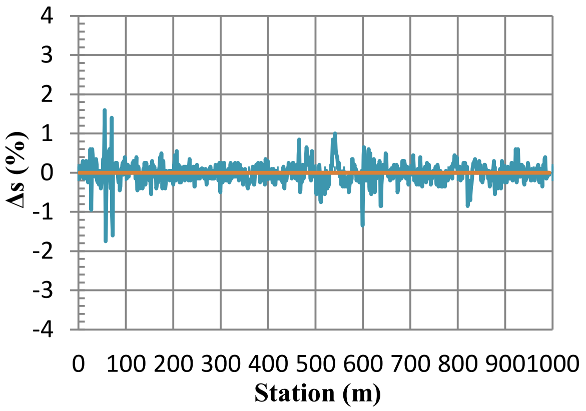


Figure 9. Road's center line's Δs [%] from the 2nd measurement with Trimble MX8, L = 2+2 m.

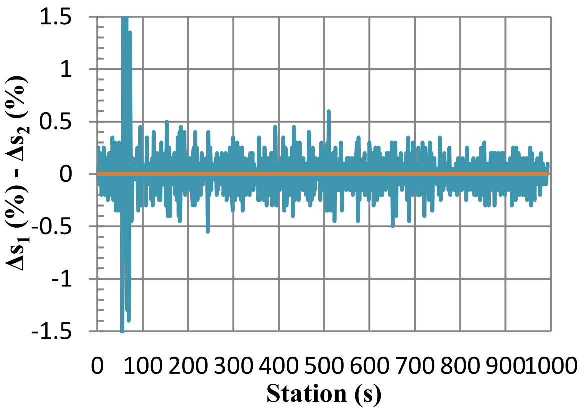


Figure 10. Differences in Δs [%] between 1st and 2nd measurement along center line with Trimble MX8, L = 2+2 m

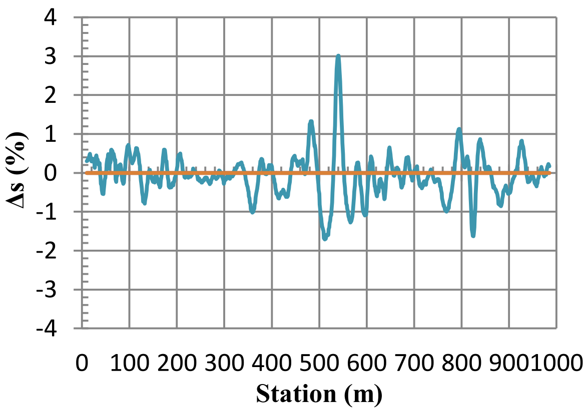


Figure 11. Road's center line's Δs [%] from the 2nd measurement with Trimble MX8, L = 10+10 m

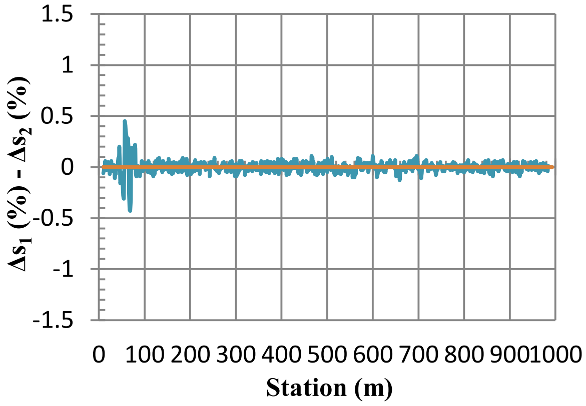


Figure 12. Differences in Δs [%] between 1st and 2nd measurement along center line with Trimble MX8, L = 10+10 m.

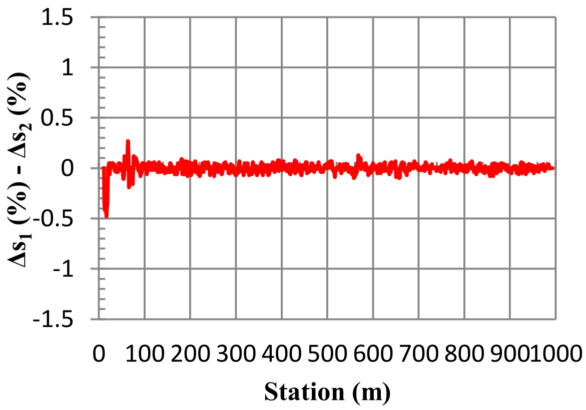


Figure 13. Differences in Δs [%] between Trimble MX8 (signalised method) and Riegl VMX-450 (bases tatiosn method) , L = 10+10 m

3.4.3 Repeatability

The correlation between two different individual measurements done with the same device was close to 100 % with a 1000 height observation data set (1 observation every meter along the centreline). Even when excluding the smallest deviations and so increasing the dispersion, the correlations were 99%.

The key figures of the distributions, such as average anomaly, were similar to the results presented in tables 1-3. One example of the distributions is presented in figure 14. The added value of the distribution calculations is that from figure 14 we can see the abnormality of the distribution. The differences between two measurements are emphasized on the other side of zero. This phenomenon recurred in all of the data sets.

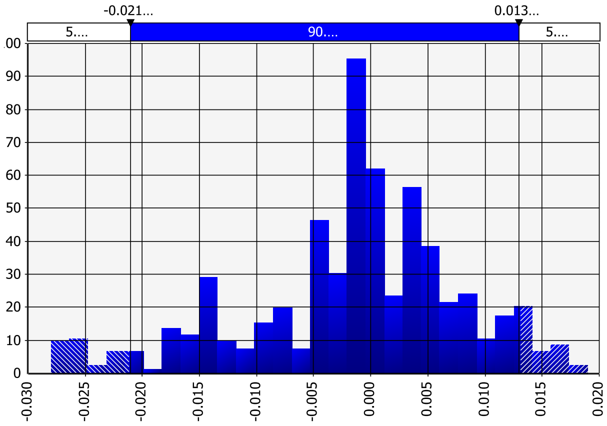


Figure 14. The distribution of height anomalies between measurements 1 and 2 from Trimble MX8

Gage-R&R tests gave generally good results. When using the whole 1000 observation data set, all of the result were <10%. If a data set of 100 subsequent heights were used, some of the results went to 30 %, but most of them were around 10 %.

4 Conclusions and discussion

4.1 z-accuracy of the surface models

The results from the height accuracy analysis were promising at least and a few clear conclusions can be made. Generally, the surface of the road can be modelled quite accurately.

Levelling should be used in the reference measurements, if possible. This is because the height differences formed between the total station and the levelling instrument. It cannot be said which of them is more accurate based in these results, but general assumption is that levelling is more accurate.

Using signals or base stations has a significant effect on the accuracy of mobile laser scanning. This can be seen by comparing figures 7-9. Especially in spot with bad satellite reception the inaccuracy of the models grows quite steeply. At reference point no. 132, were there was a crossing bridge of another road, the anomaly is about 13 cm. This is not acceptable in this type of surveying. Signals or base station are needed if the object is to measure depressions on road surface.

The quality of the satellite reception affected all of the models. The effect of the crossing bridge on station 50-100 m can be clearly seen also in the signalized models. The blind spots of the satellites should be carefully examined before the measurements and the locations of the signal patterns should be decided accordingly. For example, there should always be signal points under a bridge. Also dense forest has an effect on the positioning.

In the base station method, the fixed points have to be chosen carefully. They cannot be in blind spots because all of the positioning goes through them. If the fixed points cannot be chosen accordingly, the accuracy of the models can decrease significantly. This method is not affected so much by the other blind spots along the road. Therefore clear conclusions cannot be made of the anomalies in the base station model. Only the effect of distance to the base station is brought out. Usually the accuracy decreases farther away from the base station.

4.2 Accuracy of longitudinal slopes

One clear phenomenon could be identified throughout the longitudinal slope analysis. The better the accuracy between two measurements is, the longer the used strings (L) are. This can be seen by comparing figures 9 and 11 or 10 and 12. The longer strings also show the depression and other deformations more clearly (figure 11). The differences between two measurements are very small only with the exception of the crossing bridge. Consequently, the significance of the signal planning is emphasised also in the longitudinal slopes.

The differences between the two used devices follow the same pattern. The longer the used strings were, the better the anomalies were. Figure 13 shows this. The same bridge can still be seen in this figure too.

Generally the magnitude of the anomalies between different measurements and devices are small enough for road depression measurements in some cases. The string length used on the calculations should be carefully decided based on the quality (shape and size) of the depressions.

4.3 Repeatability

The repeatability could be established already from the graphical analysis. Generally the results of the statistical repeatability analysis were very good. However, because of the lack of resources, some important variables were not examined with enough consideration. For example, all of the analyses were made with the whole data sets of 1000 observations. Especially the Gage-R&R test was then tried out with 100 observation data sets. Even this was not exactly the right way to do it. The right way would have been to randomly pick 100 observations from the 1000 and compare them. This would have removed the effect of autocorrelation. The one result of 30 % is a clear indication of autocorrelation. If one result is bad, others that will follow are also going to be bad.

4.4 Further development and research

Although the results were promising, some further definitions and research is also needed. For example, the accuracy of the heights and slopes should be examined locally, in 50 m and 100 m segments. The accuracy would be worse this way because of the properties of satellite positioning.

The possibilities of the mobile laser scanning technology and extensive 3D-models should be examined and utilized more effectively. The method for calculating the depressions was the same that it has been for decades. Only the source of the surveying data was different.

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