GEODETIC MONITORING METHODS OF LANDSLIDE-PRONE REGIONS – APPLICATION TO RABENOV

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ABSTRACT

The article focuses on the geodetic monitoring of the Rabenov landslide territory. The monitored slope is situated in the landscape deteriorated by mining activity in North Bohemia, West of Ústí nad Labem, in Chabařovice, a reclaimed brown coal pit. The article presents a general description of the application and accuracy of geodetic methods used to measure on the movement of material in landslides. Also, the history, the problems and the plans for reclamation of the monitored territory are briefly mentioned. The text concentrates on the description of the network of three observed points which are formed by geotechnical instrumentations and on the description of a terrestrial and GNSS measurement in the network of monitored points. The assessment methods and the results from the eight-year phased monitoring process are also discussed. Different methods were tested during the phases. For terrestrial measurements, standard deviations of about 2 mm in position were made and for GNSS standard deviations of about 14 mm. Surveys in a local grid showed that there were continuous movements of observed station points of about 10 mm per year. The paper also discusses the use and assessment of digital terrain models in zones with on-going artificial as well as natural changes. Models indicate elevation changes of about 2–6 m and movement of about 10,000–100,000 m^3 of material.

Keywords: digital terrain model (DTM), geodetic monitoring, global navigation satellite systems (GNSS), Rabenov landslide territory, terrestrial measurements

1. Introduction

The monitoring of landslide territories is a multi-disciplinary branch which, based on systematic phase measurements, observations and analyses of the current state, leads to the estimations of the slope development, the designs of safety measures and potentially the documentation of past landslides. The monitoring is implemented in areas where the occurrence of shifts and deformations is probable or has already been manifested. Landslides cause natural disasters or limit the territorial development in many countries worldwide being the subject of great research interest (Pesci et al. 2004; Burda et al. 2013).

In monitoring landslides, geodetic methods are applied together with geotechnical methods using, among others, inclinometers, declinators, extensometers and hydrostatic levelling instruments to measure deformations, stresses and forces. Geotechnical measurements are costly requiring the digging of deep trial holes for measurements; the results are data on spatial deformations in individual layers of soils representing, however, local relationships. In the case of insufficient coverage of the monitored territory by geotechnical probes, additional methods must be used such as surveying, which contributes to the prediction of the depth and extent of movements. Geodetic methods may provide data on the overall deformations of the monitored area of interest and, apart from measurements (cubatures, longitudinal profiles, mapping of soil shifts within the whole territory, monitoring of stabilised survey marks), they also focus on the coordinate description of positions of geotechnical facilities.

The expansion of new technologies has allowed the development of novel measurement techniques as well as innovations of long-term proven methods applied in failure zones. The text below presents geodetic methods that are generally applicable for the monitoring of landslides and specific applications for the long-term monitored Rabenov landslide territory in North Bohemia.

2. Geodetic monitoring methods

Geodetic monitoring methods of slope movements may be subdivided into "point" and "area".

In point methods, specific points (marks) are monitored which are usually formed by geotechnical probes with mounted reflective targets or otherwise suitably stabilised points. If the spatial position of monitored points changes between individual measurement phases, the vectors of coordinate changes are non-zero being greater than the identified difference limit value expressing the measurement inaccuracy, and the change is called a shift. A shift may be "relative", if it is related to other observed points, or "absolute", if related to the grid reference system. Depending on the comparison with the other points, conclusions and assumptions about the slope behaviour must be formulated. Point methods may have little conclusive evidence if the coverage of the whole territory is incomplete, covering only certain parts of it is guaranteed. These methods include classical terrestrial measurement using total stations or levels and methods using global navigation satellite systems (GNSS).

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Research terrestrial measurements are usually used in geodetic networks where the accuracy of the spatial coordinates ranges in millimetres (local network Rabenov). Classical terrestrial measurements from one standpoint are used e.g. in ČSA Pit where 45 permanently stabilised points are monitored at regular hourly intervals (Stanislav, Blín 2007) with the accuracy rate of the spatial coordinates identification of 20–30 mm (Hampacher et al. 2008).

The measurements with GNSS are easier than terrestrial methods, but the accuracy of position mainly depends on the observation time, atmosphere conditions, unobstructed view of the sky, number of satellites and transformation of coordinates into reference system (Raška, Pospíšil 2011; Urban et al. 2013). The position of observed points with permanent stations may be determined with millimetre accuracy (Manetti et al. 2002). The expected accuracy of the coordinates of points with rover stations and with the observation time of ca. 20 minutes (static method) may be ca 5–10 mm and in mapping the real time kinematic (RTK) method (measurement of 5 s), the expected accuracy is of ca 25–50 mm.

In "area" methods, as their name suggests, the entire monitored territory is surveyed where guided by the principles of mapping the measured points are selected on prominent terrain landmarks, on the very faults and within a regular grid or the points are obtained by an automated survey technology. Such measurements usually allow plotting a digital terrain model (DTM), which represents the measured surface with its morphological features and enables the observer to make a complex overview of the monitored area of interest and changes occurring in it. DTM may subsequently serve for making various analyses (changes in volumes, shifts of soil, changes in slopes), or for exporting longitudinal profiles and contour plans. An extensive number of measured points may be obtained by using classical measurement methods with total stations (the polar method), RTK GNSS technologies where points are selected by the operator, or automated methods using laser scanners or photogrammetry.

Laser scanning is an automated method where the instrument measures separately in a certain preset steps (ca 10 cm onto 100 m). The result of such measurement is a point cloud with the accuracy of coordinates of ca 20–30 mm (Pospíšil et al. 2006). It is a developing method where the measurement conditions and the accuracy of shifts are studied in many theses (Barbarella et al. 2013; Abellán et al. 2009).

Photogrammetry is based on the identification of coordinates from photographs. The accuracy of the resulting points is ca 10 cm, and the method is comparable to laser scanning. The advantage of this method is the speed of taking the image (capturing) of a monitored object with minimum regards to its shape complexity and inaccessibility. Apart from terrestrial photogrammetry, aerial photogrammetry may expediently be used for extensive landslide bodies; the results are orthophotos of the territory and DTM (Pesci et al. 2004).

Apart from the classification by the data collection methods, geodetic monitoring may also be classified according to the measurement interval and assessment into "active" and "passive".

In active monitoring, automated total stations performing automated measurements according to the preset programme or permanently installed GNSS receivers may be applied. These instruments make measurements in short time intervals and immediately send the results into the monitoring centre connected to the hazard reporting system. Such measurements are only made in high risk zones. An example may be the ČSA Pit and the monitoring of South East slopes of the Krušné Mountains (Burda et al. 2010).

In passive monitoring, the interval between individual measurement phases depends on the hazard factor and the client's requirements ranging from days to years. The results of measurements are only known after they are assessed in the office by a surveyor, and the outputs are usually extensive reports including the assessment of observed points and summary maps of the monitored territory. Passive monitoring is carried out on the described Rabenov locality or elsewhere in the world (Bitelli et al. 2004; Barbarella et al. 2013).

3. Rabenov landslide territory

The monitored Rabenov landslide territory is a part of outer dumps of the former Chabařovice opencast brown coal mine situated 5 km to the southwest of the town of Ústí nad Labem (Figure 1). The slope is situated to the southeast of the extracted pit space, below the peak of Rovný Hill (376 m.s.l.). The upper South part, so-called top site, was not directly affected by overburden, but has impaired stability thanks to the reduction of load and the removal of the foot of the slope. The lower North part, the bottom site, is formed by an unstable outer dump. The monitored area covering about 40 ha is situated between elevations 145 and 275 m.s.l. The dimensions of the territory are 1.15 km × 0.37 km (0.30 km at the foot of the slope).

In Figure 1, the whole monitored territory is marked with a subdivision into subterritories. The upper South (A) and the lower North (B) part cover the entire monitored zone, and the border between them is the road running across the centre of the territory at an altitude of ca 190 m.s.l. The other two parts have recently become the focus of interest due to extensive changes. Landslides continuously occur in the surface soil layer in the southeast part (C), while extensive material transfers (130,000 m^3 of stone material) were performed at the foot of the slope (D) (former spoil tip of titanium clays (PKÚ website, 2013)) to ensure the slope's stability.

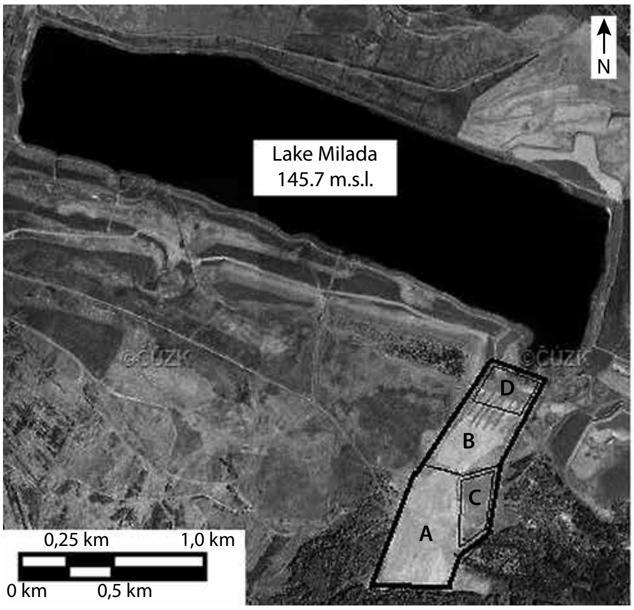
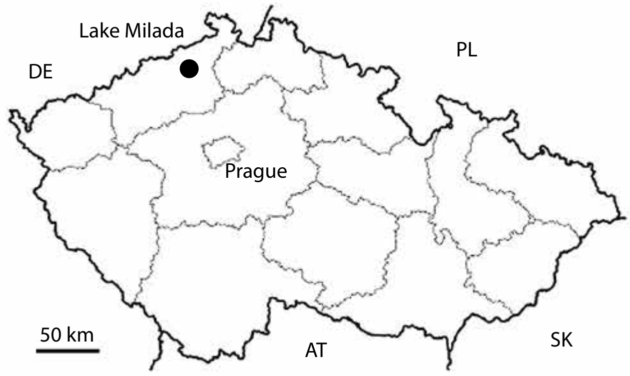


Fig. 1 An orthophoto with a marked monitored territory.
Source: Braun 2011



3.1 History of the territory

Chabařovice Pit was one of the last opencast mines in the North Bohemian brown coal basin. The new pit was situated in a flat valley of the Modlanský Stream where outer dumps were established on southern slopes and mining was carried out northwards along less steep slopes. Mining began in 1977 and it was presumed that the open pit operation would be terminated after the extraction of all coal deposits in the easternmost part of the basin. In 1991, the Government Decree of the Czechoslovak Federal republic decided to stop mining in Chabařovice Pit, and a strict mining policy was set up. This decision was caused by an attempt to preserve the town of Chabařovice and adjacent steelworks. The actual shutdown began in 1994, and all mining, processing and sales of coal ended in April 1997. In March 2000, the last technological unit, ensuring the backfilling of the bottom of the residual pit with soil according to the approved shutdown plan, was stopped.

The total of 61.5 million tons of high-quality low-sulphur brown coal, 9.3 million m^3 of spoiled material and 256.1 million m^3 of overburden were extracted during the mining activity in Chabařovice Pit. After mining was stopped, additional 128 million tons of coal, which were supposed to be extracted according to original plans, were left in the deposit (Šípek, Němec 2008).

At the time of extraction, the monitored Rabenov slope was used as an outer dump for storing overburden soil. Local landslides of uncovered layers of clay were

Fig. 2 Landslide in the southeast part of Rabenov (April 2009).
Source: Braun 2011

manifested on the Rabenov slope territory as early as the 1990's. Stability problems were partly caused by deposits of non-cohesive soils (loess, soils with high contents of boulders), but mainly by wetting of the area originating from former small water supply structures. As the stability design using a mining method (supporting by layers of an interior dump) was technically not feasible due to the limitation of mining, a decision was made in 1994 to carry out remedial works using a construction method. In 1994–1999, the state-owned Palivový kombinát Ústí (PKÚ) took measures to secure the slope, including, in particular, the construction of drainage structures and the foundation of supporting stabilization ribs. Despite the efforts to stabilize the territory, slope movements, that are even now apparent in the southeast (C) part of the slope, developed over time (Figure 2).

3.2 Restoration and planned use of the territory

In April 1999, the Ministry of the Environment of the Czech Republic approved the "General Plan of Land Reclamation until the Completion of Complex Revitalization of the Territory Affected by Mining Activity by the State-Owned Palivový kombinát Ústí". Based on this plan, reclamation works are currently in progress in the area performed by the Palivový kombinát. Planned completion date of the work is after 2015.

The surface area of the territory disturbed by mining and by founding outer dumps is nearly 1500 ha. The basic concept of remediation and reclamation activities, which are aiming to restore the landscape function in the area disturbed by mining, is the hydric reclamation method of the residual mining pit, i.e. its filling with water. The creation of the total of eight lakes in residual mining pits is planned in the North Bohemian brown coal basin. The hydric reclamation process was first used in Chabařovice Pit, which became a prime model site in terms of a potential future application of the knowledge gained during reclamation activities in the other 7 residual pits.

The filling of the newly created Milada Lake (formerly known as Chabařovice Lake) with water from streams in the Krušné Mountains and water from the residual pit's watershed started on the 15th of June 2001. The estimated filling time was 5–6 years. On the 8th of August 2010, the filling was terminated by reaching the final operating water level at an elevation of 145.7 m.s.l. After reaching this level, the lake covers an area of 252.2 hectares, retains 35.601 million m^3 of water with the maximum depth of 24.7 m and an average depth of 15.5 m (PKÚ website, 2013).

The southern part, which accommodates the monitored area and consists mainly of outer dumps, is planned to fulfill primarily ecological functions. Forest reclamation will be complemented by new grassed areas. In the southeast part of the lake adjacent to the slope, a bay was formed to be used as a jetty for boats (sailboats, barges, sports boats).

Remediation work on the monitored Rabenov slope began in 2006. Its main objective is to secure the geomechanical stability of the slope. This is done by 7 anchored pile walls (Figure 3) installed in the southern part at an elevation of 240 m.s.l. in 2007, by building drainage structures, by the removal of unstable soil layers along with the modification of the gradients of parts of the slope and by the building of stabilization benches at the foot of the slope.

4. Geodetic monitoring of Rabenov landslide territory

Guided by the above described reasons, the monitoring of the Rabenov landslide territory was performed by the Department of Geotechnics and the Department of Special Geodesy of the Faculty of Civil Engineering, CTU in Prague working for the state-owned Palivový kombinát Ústí, which is in charge of land reclamation. Three instrumented inclinometric boreholes for geotechnical measurements, which were assumed to extend into stable underlying rock, were bored. They were fitted with combined casing to a depth of 24 m, and hydrological probes for the monitoring of the water table were installed. Geodetic monitoring running in a mutual coordination with

Fig. 3 Distribution of retaining walls in the upper South part of the slope (April 2010).
Source: Braun 2011

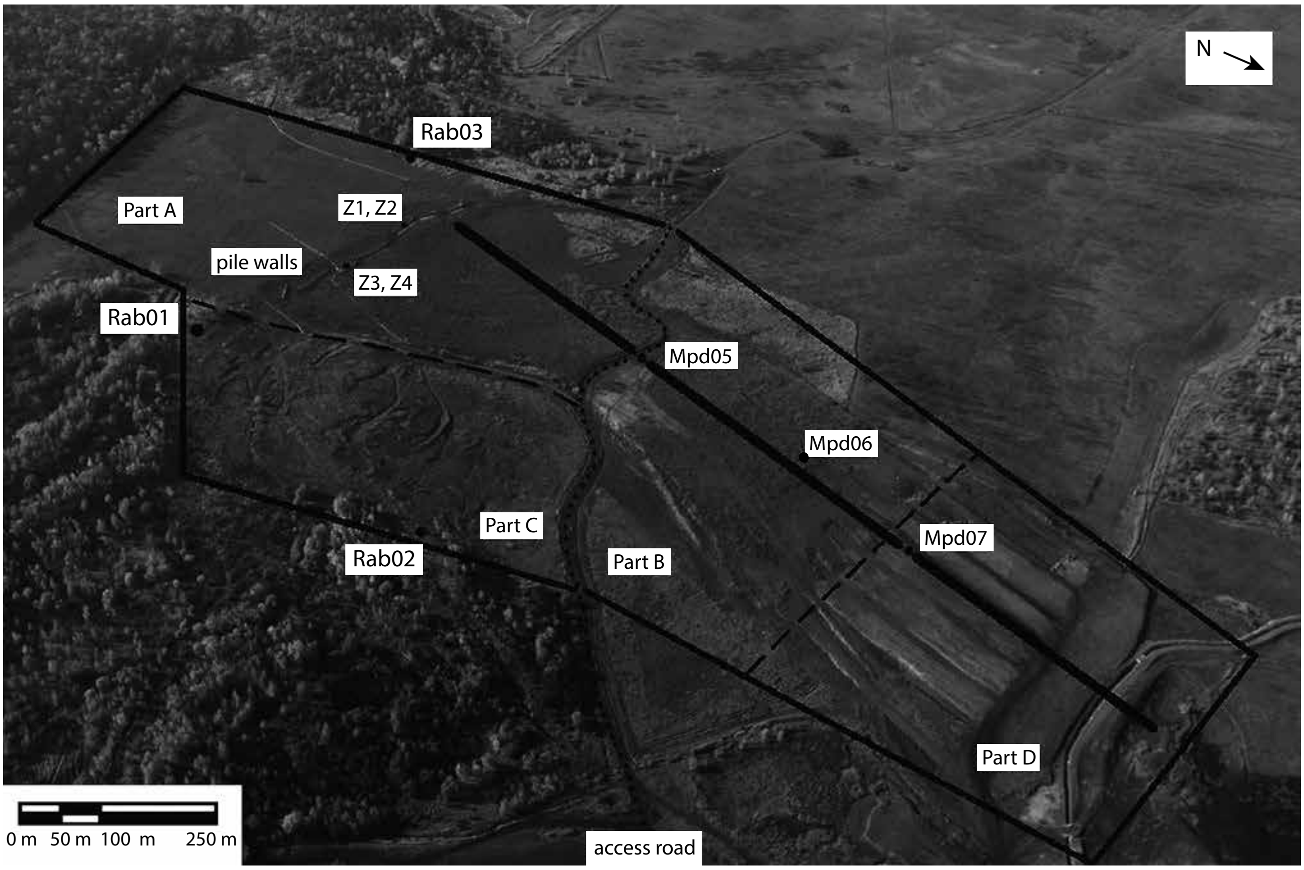


Fig. 4 Aerial view of Rabenov slope with marked points and the profile route (October 2008).
Source: Braun 2011

geotechnical monitoring was performed since 2003 to 2011, including 19 measurement phases in all. In 2003 and 2004, the measurements were done three times a year (in April, June and October). From 2005, the measurements were performed twice a year (in April, July). Various monitoring and assessment methods for the identification of changes of points and the slope were tested during individual phases.

Individual phases involved the survey of a local spatial grid and the monitoring of detailed survey points using terrestrial methods (in 17 phases) and the GNSS technology (in 12 phases). Another element of the monitoring survey was the establishment of monitoring longitudinal profiles (in the last 7 phases) and the survey of background data for the generation of digital terrain models (in the last 5 phases). All of the results were presented in final diploma theses at the Department of Special Geodesy, e.g. (Braun 2011; Rytíř 2012; Riegerová 2012). Figure 4 displays the main observed points and the route of the monitored longitudinal profile.

4.1 Terrestrial measurement – local spatial grid

The establishment of the Rabenov local grid in the southern part of the slope in 2002 was part of the GA ČR grant project No. 103/02/116 "Research and verification of slope movement monitoring methods".

The Rabenov spatial survey grid was originally composed of four points. Points labelled Rab01, Rab02, Rab03 are formed by instrumented inclinometric boreholes. Spatial deformations of the earth body in the borehole are measured with a modified inclinometer (horizontal changes) and the sliding deformeter (vertical changes). The borehole head is modified to use a special centering fixture which serves for an unambiguous final centering of surveying instruments on the tripod. The borehole head also houses a bench mark to which the point's elevation is referred. The fourth station point labelled Rab04 was a chaining arrow located on a concrete base of an old unused power line pole in the upper part of the slope. 23 discrete characteristic points of the terrain were observed from the points of this grid. These points were stabilised by a steel rod with a diameter of 0.06 m and a length of 1.25 m with an internal thread at its upper end into which a special fixture with two all direction reflective prisms had been mounted (Hánek 2007). Point Rab04 is presently no longer used for a measurement (since 2007), and the original detailed survey points are no longer surveyed either for the reason of their destruction during earthworks or their disappearance due to the effect of growing natural self-seeding vegetation (since 2008). Starting from the 12th phase (April 2008), points Z1–Z4, which are formed by geomechanical instrumentations on two pile walls, have been surveyed from the points of the grid. The instrumentations were installed for the measurement of relative deformations of the reverse and the face of the bent pile by means of a sliding micrometer and for the identification of the wall activation pattern due to the pressure of the stabilised slope (Záleský et al. 2013a). These points are used for the identification of the centre of the protective lid with the main emphasis on the points' elevation. 7 pile walls were built for the slope stabilisation in 2007 based on the results of on-going monitoring.

The starting point of the local grid is Rab01, which is considered stable for calculations. The Rab01–Rab03 connecting line, 419 m in length, lies approximately on the horizontal, on a convex fault of the terrain, and the +X axis of a local coordinate system is laid onto it. The height difference of Rab02–Rab03 points reaches ca 54 m for a length of 664 m, and the height difference of Rab02–Rab01 points reaches ca 52 m for a length of 364 m. The grid has a shape approximating an isosceles triangle. The distribution of points is displayed in Figure 5.

To ensure a highly accurate survey of the standpoints of the grid and the detailed survey points, Leica TC1700 (TC1800) total stations, which have the manufacturer declared accuracy of 0.5 mgon (0.3 mgon) in angles and 2 mm + 2 ppm in lengths (ISO 17123-3 2001 and ISO 17123-4 2001), were used in measurement phases 0–15. In phases 16–18, more up-to-date Topcon GPT7501 instruments with the manufacturer declared accuracy of 0.3 mgon in angles and 2 mm + 2 ppm in lengths (ISO 17123-3 2001 and ISO 17123-4 2001) were used. The measurements were performed with targeted prisms matching the used instruments and also with all direction Leica prisms, which accelerated the targeting from different standpoints during the measurement onto detailed survey points. The grid survey always followed the procedure in which tripods were centered and levelled over the standpoints, and then horizontal angles, zenith angles and slope distances were measured from each standpoint onto other standpoints and visible detailed survey points. All measurements were performed minimally in two rounds.

As a redundant number of variables were always measured in the grid, the least squares adjustment method was used for the calculation of individual spatial coordinates.

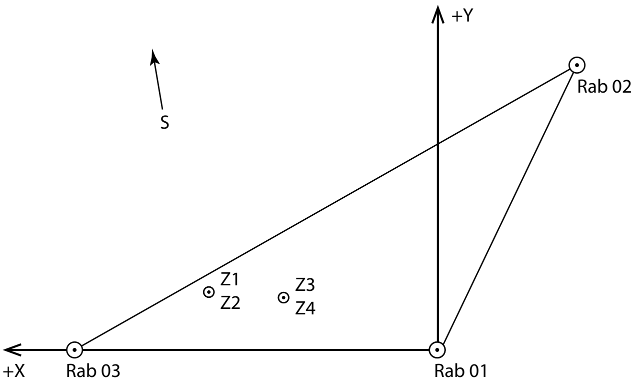


Fig. 5 A sketch of the Rabenov local grid.
Source: Braun 2011

The method of free net was chosen with one defined point (Rab01) and direction of X-axis. The resultant accuracy of the coordinates ranges around 2 mm thanks to accurate instruments and the selected measurement procedure. The detailed measurement and calculation procedure is presented e.g. in (Braun 2011).

Based on the resulting coordinates, phase changes between individual points may be compared, in particular a change in horizontal length and in height difference. As the grid only contains of 3 points of which point Rab01 was assigned fixed coordinates, the assessment of changes cannot be made using solely the differences in coordinates between phases.

Despite this drawback, a shift of 0.04 m in the slope direction and of −0.01 m in height was manifested in the standpoints from the measurement in 2003–2008. Over the years, the slide slowed down and only the slope settlement continued.

Because of the unsuitable grid configuration and the minimum number of points, potential unstable standpoints were identified using the method of comparing horizontal lengths and height differences between individual points. This procedure is based on the assumption that one of the points reaches greater changes in position than the other two points, and the length (height difference) does not contain this point, changes least of all between phases during the comparison of three lengths (height differences).

The difference limit value between phases was identified using a general law on the accumulation of standard deviations, and based on the results of grid adjustment (standard deviations of coordinates) in phase 16 and 17. The difference limit value in horizontal length between phases was identified as 5 mm, and the difference limit value in height difference between phases as 3 mm.

The horizontal lengths (D) between standpoints in the last seven phases are presented in Table 1, together with differences against phase 0 ($\Delta D_{0,i}$) and differences between individual phases ($\Delta D_{i,i+1}$). Based on the assumptions above, horizontal shifts are still likely to continue, particularly on point Rab02.

The height differences (dH) between station points from the last seven phases are presented also in Table 1, together with differences against phase 0 ($\Delta dH_{0,i}$) and differences between individual phases ($\Delta dH_{i,i+1}$). Based on the assumptions above, shifts in elevation are likely to occur on point Rab03.

The changes in length and height difference are also displayed in charts (Figure 6–11). The table and charts also imply that changes between phases grew smaller and, therefore, the slope stability improved after the installation of piled retaining walls in 2007. At the same time, some periodicity in height changes (ca 30 mm per season) of point Rab03 may be observed, which probably relies on the season (water saturation of the soil).

Detailed survey points Z1–Z4 on retaining walls were verified in the same way as standpoints. The main

Tab. 1 Lengths and Height differences between standpoints and their changes.

Phase	Points	D	$\Delta D_{0,i}$	$\Delta D_{i,i+1}$	dH	$\Delta dH_{0,i}$	$\Delta dH_{i,i+1}$
		[m]	[mm]	[mm]	[m]	[mm]	[mm]
0 (04/2003)	Rab01–Rab02	364.162	–	–	52.279	–	–
	Rab01–Rab03	419.043	–	–	1.854	–	–
	Rab02–Rab03	664.196	–	–	54.133	–	–
12 (04/2007)	Rab01–Rab02	364.200	−38	8	52.276	3	−8
	Rab01–Rab03	419.055	−12	7	1.787	67	−24
	Rab02–Rab03	664.255	−59	13	54.063	70	−32
13 (08/2008)	Rab01–Rab02	364.204	−43	−5	52.261	18	15
	Rab01–Rab03	419.056	−13	−1	1.753	101	34
	Rab02–Rab03	664.265	−70	−10	54.014	119	49
14 (04/2009)	Rab01–Rab02	364.211	−49	−6	52.274	5	−13
	Rab01–Rab03	419.079	−36	−23	1.771	83	−18
	Rab02–Rab03	664.291	−95	−26	54.045	88	−31
15 (07/2009)	Rab01–Rab02	364.219	−57	−8	52.275	4	−1
	Rab01–Rab03	419.083	−40	−4	1.751	103	20
	Rab02–Rab03	664.312	−116	−20	54.026	107	19
16 (04/2010)	Rab01–Rab02	364.203	−41	16	52.267	12	8
	Rab01–Rab03	419.057	−14	26	1.771	83	−20
	Rab02–Rab03	664.270	−74	41	54.038	95	−12
17 (07/2010)	Rab01–Rab02	364.197	−36	5	52.266	13	1
	Rab01–Rab03	419.054	−11	3	1.745	109	26
	Rab02–Rab03	664.272	−76	−2	54.011	122	27
18 (07/2011)	Rab01–Rab02	364.197	−35	0	52.264	15	2
	Rab01–Rab03	419.050	−7	4	1.776	78	−31
	Rab02–Rab03	664.264	−68	8	54.040	93	−29

Source: Braun 2011, Rytíř 2012

monitored parameter was the height difference between points, where the difference limit value in height differences between two phases is identified at a value of 30 mm (based on adjustment results and experience with measurement). The higher value of the difference limit value was accepted as the measured points are represented by a screwed-in lid and no control was done whether the lid has always been identically screwed in between phases, plus there is some inaccuracy in the identification of the prism height on the detail pole. Nearly all differences between phases presented in Table 2 are smaller than the identified difference limit value, except for the difference between phase 14 and 15 on points Z2, Z4. The value exceeds the difference limit value by only 3 mm, besides, a change due to the screwing of the lid may be suspected there. Based on the results, no changes between piled retaining walls were manifested, which also confirms the functionality of these measures for enhancing the slope stability.

4.2 Terrestrial measurements – assessment of detailed survey points

The accuracy in the determination of the spatial position of observed points of the terrain and the centres of probes is described in the formula for the standard deviation of the 3D polar method, considering the point survey method using a fixture with a mounted pair of all direction prisms. During the measurement of a detailed survey point, the fixture is screwed in the detailed survey point's stabilisation and it may be said, that the axis of the fixture is the extension of the point stabilisation axis. Knowing these axes (vectors) the angle formed by them between individual phases may be calculated using analytic geometry. The changes identified in the inclinations of survey marks may subsequently be used to draw conclusions about the pressures acting in the surface layers of rock. In stabilised detailed survey points, therefore, not only changes (shifts) in coordinates, but also changes in the inclination of the stabilisation may be monitored. The issues of determining the inclination of survey marks are treated in more detail in (Bubeník et al. 2006).

Stabilised detailed survey points in the locality were surveyed from 2003 to 2006. The average monthly shift of observed points measured on the Y axis was +10 mm to +49 mm, on the X axis −7 mm to +4 mm, and on the Z axis −11 mm to −36 mm. It is obvious that the greatest shift in position occurred on the Y axis, while it is almost zero on the X axis. This may be explained by the

Tab. 2 Height differences between detailed survey points and their changes.

Height differences between detailed survey points			**Differences against phase 12**	**Differences against previous phase**
Phase	**Points**	**dH [m]**	**$\Delta dH_{12,i}$ [mm]**	**$\Delta dH_{i,i+1}$ [mm]**
12 (04/2007)	Z1–Z3	2.948	–	–
	Z2–Z4	2.964	–	–
13 (08/2008)	Z1–Z3	2.964	−16	−16
	Z2–Z4	2.966	−2	−2
14 (04/2009)	Z1–Z3	2.952	−4	12
	Z2–Z4	2.936	28	30
15 (07/2009)	Z1–Z3	2.959	−11	−7
	Z2–Z4	2.969	−5	−33
16 (04/2010)	Z1–Z3	2.953	−5	6
	Z2–Z4	2.967	−3	2
17 (07/2010)	Z1–Z3	2.967	−19	−14
	Z2–Z4	2.975	−11	−8
18 (07/2011)	Z1–Z3	2.959	−11	8
	Z2–Z4	2.969	−5	6

Source: Braun 2011, Rytíř 2012

orientation of the axes where the +Y axis is almost identical to the terrain fall line, and the +X axis lies horizontally.

The VZ MSM 6840770001 research plan – "Reliability, optimisation and durability of building materials and structures" involved the construction of a mathematical model allowing the assessment of deformations of detailed survey points. The mathematical model is based on the application of fuzzy logic which is presently used in numerous branches for intelligent control systems. The advantage of the model applying fuzzy logic is the possibility of including also data that cannot be considered in the classical concept in the decision making process of the point's shift. For, fuzzy logic allows us to include, apart from objective, unambiguously measurable views, also subjective views in a statement on deformations. Then, the representatives of objective views in the generated model are the spatial standard deviation of an identified point in individual phases of measurement, the point stabilisation method, the relevance of the observed point, the error of automated targeting of the instrument or the operator's personal error and climatic conditions during the measurement. Subjective views are represented by the impressions of the participants in the measurement – visual control of the terrain in close proximity to the point and its assessment, the operator's feeling during focusing on the target (Note: targeting may be made more difficult by climatic or vegetation conditions). Based on input parameters defined in this way, the decision making mechanism may be set up containing several fuzzy rules, after its defuzzification the observed points may be divided into several categories expressing their stability. The advantage of the model is a possibility of generating scales of point categories of different fineness meeting the needs of the recipients of geodetic monitoring results against the commonly used two degree scale. A finer categorisation may bring advantages during the interpretation and, in particular, the application of survey results as input data for the generation of prediction or assessment models of phenomena in monitored localities. The whole set-up of an experimental model is described in (Hánek, 2009) and its brief summary can be found in (Hánek, 2010).

4.3 GNSS measurements

Starting from phase 3 of April 2008, the measurement of standpoints using the fast-static GNSS method began with the objective of enabling the grid connection to the national reference systems (connection by the terrestrial method would be less feasible) and thus identifying which points move and in which direction. The Czech reference systems are S-JTSK (map coordinate reference system) and Bpv (Baltic vertical datum – after adjustment).

The GPS Trimble 5700 apparatus was used between phase 3 and 17, and the fast-static method with the observation time of 8 minutes onto minimally 6 satellites of the NAVSTAR-GPS system and double measurement on identified points was selected. In phase 18, three Topcon HiPer Plus receivers and one Topcon PG-A1 receiver were used, and experimental measurement on multiple stations was simultaneously performed with different observation times. The results of this experiment are presented in (Riegerová 2012).

Until phase 17, the value of 14 mm was considered the standard deviation in position σ_{p17} and in height σ_{H17} (based on the results of the calculations), and the difference limit value in position and height of a point between phases of 49 mm (a general law of the accumulation of standard deviations). The standard deviation in position σ_{p18} for phase 18 was determined from the resulting coordinates in the value of 13 mm, and the standard deviation

Tab. 3 Comparison of coordinates – Differences in coordinates against phase 3.

Point	Rab01			Rab02			Rab03		
Phase	$\Delta Y_{3,i}$ [mm]	$\Delta X_{3,i}$ [mm]	$\Delta H_{3,i}$ [mm]	$\Delta Y_{3,i}$ [mm]	$\Delta X_{3,i}$ [mm]	$\Delta H_{3,i}$ [mm]	$\Delta Y_{3,i}$ [mm]	$\Delta X_{3,i}$ [mm]	$\Delta H_{3,i}$ [mm]
3 (04/2004)	0	0	0	0	0	0	0	0	0
4 (07/2004)	15	0	25	55	−20	91	−14	−47	17
5 (10/2004)	−23	−1	−38	70	−34	25	9	−67	−11
6 (07/2005)	24	−2	−36	68	−50	55	−4	−65	−24
7 (11/2005)	37	−27	−44	80	−31	25	7	−79	−27
8 (04/2006)	34	−14	−23	75	−47	31	13	−84	−28
9 (07/2006)	31	−8	−54	74	−40	8	12	−81	3
10 (04/2007)	31	−18	−43	72	−52	20	18	−97	−27
12 (04/2008)	30	−19	−12	76	−44	19	19	−100	−3
13 (08/2008)	19	−27	−39	63	−39	−76	20	−89	−22
15 (07/2009)	22	−4	−59	54	−48	35	−4	−108	51
17 (07/2010)	−27	−29	−11	21	−76	12	−46	−155	22
18 (07/2011)	12	30	2	50	−7	77	0	−77	29

Source: Braun 2011, Riegerová 2012

Tab. 4 Coordinates of points in S-JTSK and Bpv (the first and last phase).

Phase	3 (04/2004)			18 (07/2011)		
Point	Y [m]	X [m]	Z [m]	Y [m]	X [m]	Z [m]
Rab01	766400.858	977584.680	253.251	766400.846	977584.650	253.249
Rab02	766195.735	977553.771	201.035	766195.685	977553.778	200.958
Rab03	766815.303	977792.946	255.047	766815.303	977793.023	255.018

Source: Braun 2011, Riegerová 2012

in height σ_{H18} was identified in the value of 30 mm, while the difference limit value between phases is 48 mm for position and 83 mm for height.

Table 3 presents differences in coordinates against the initial phase, based on them we may conclude that the difference limit value for both position and height was not exceeded in the majority of cases. If, however, it was exceeded, it might have been due to a gross error during the measurement or processing, or the cause may be in one of the explanations below. A significant difference in position and height was first discovered between phases 3–4, which may be the consequence of building activity in the vicinity of the 000906080130 trigonometric point, which served as a reference station at that time. Another significant change occurred between phase 15–17 and phase 17–18. The most likely reason may be the use of another reference station in the calculation of coordinates. For phases 3–15, the reference station was the 000906080130 trigonometric point, while for phase 17 the reference point was a virtual station provided by the CZEPOS service. The considerable exceeding of the difference limit value $\Delta metp18$ between the last two phases may again be caused by the use of another reference station during the calculation. The reference station for the last phase, phase 18, was the permanent station of the CZEPOS network – Litoměřice (CLIT). Despite this, however, unstable points in the grid cannot be identified with due reliability (Riegerová 2012).

In analogy to terrestrial measurement, horizontal lengths and height differences between standpoints are compared in the GNSS measurement, too. The difference limit value in length applied until phase 17 was 70 mm, and for phase 18 it was 68 mm. The height difference limit value applied until phase 17 was 70 mm, and for phase 18 it was 117 mm. The difference limit value magnitudes imply that the GNSS measurement in the selected configuration is less accurate and comparing the phases the difference limit value was mostly not exceeded, thus the shift of any point was not confirmed. Figures. 6 to 11 present changes in lengths and height differences together with the results of terrestrial measurement. It is apparent that comparing lengths, the terrestrial measurement corresponds to the GNSS measurement, but comparing height differences the results between the methods are significantly different (which may also be expected due to a greater standard deviation in the height determination in the GNSS measurement).

4.4 Monitoring longitudinal profile

The route of the longitudinal profile runs along the axis of points Mpd05 and Mpd07 (Figure 4), and it was

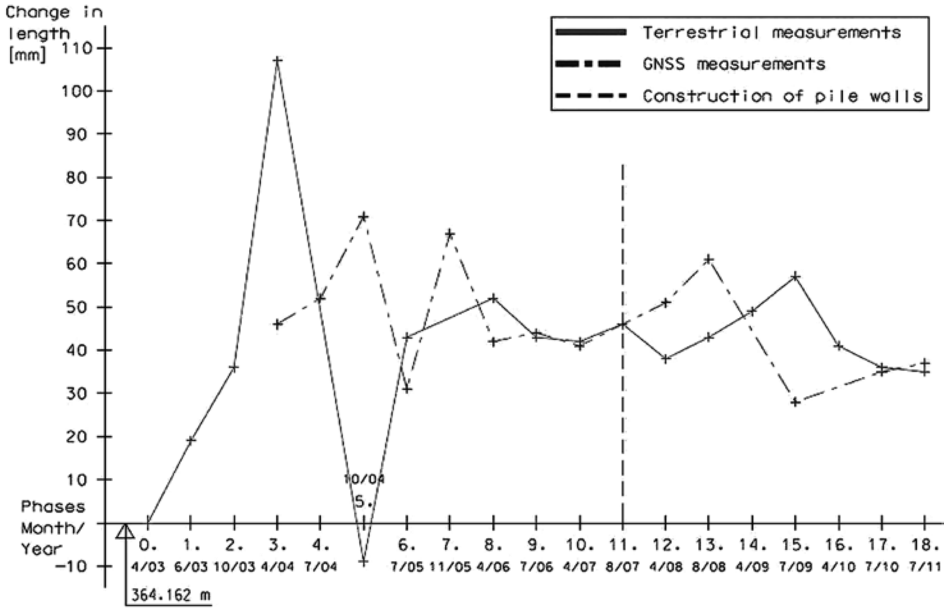


Fig. 6 Changes in length between Rab01–Rab02 points. Source: Rytíř 2012, Riegerová 2012

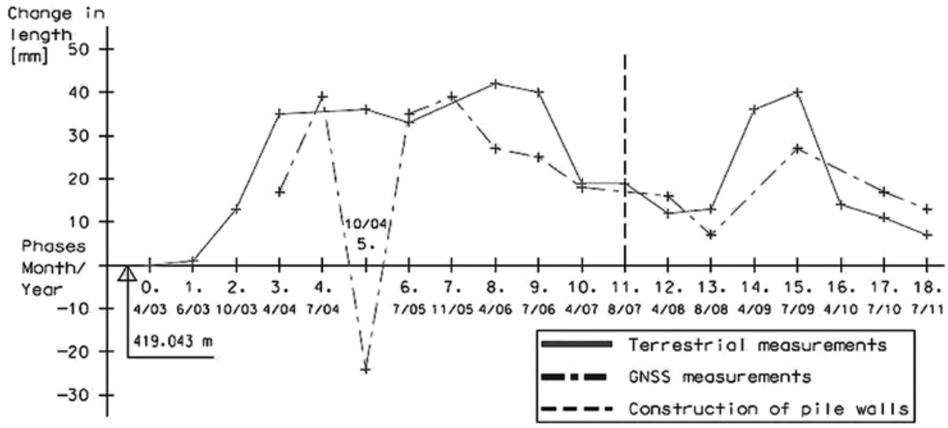


Fig. 7 Changes in length between Rab01–Rab03 points. Source: Rytíř 2012, Riegerová 2012

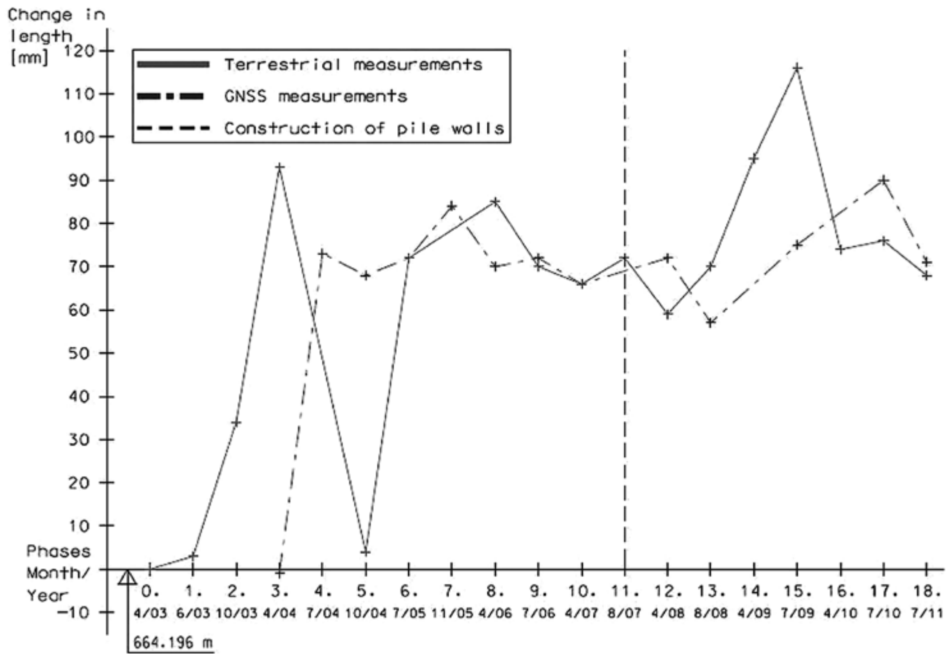


Fig. 8 Changes in length between Rab03–Rab02 points. Source: Rytíř 2012, Riegerová 2012

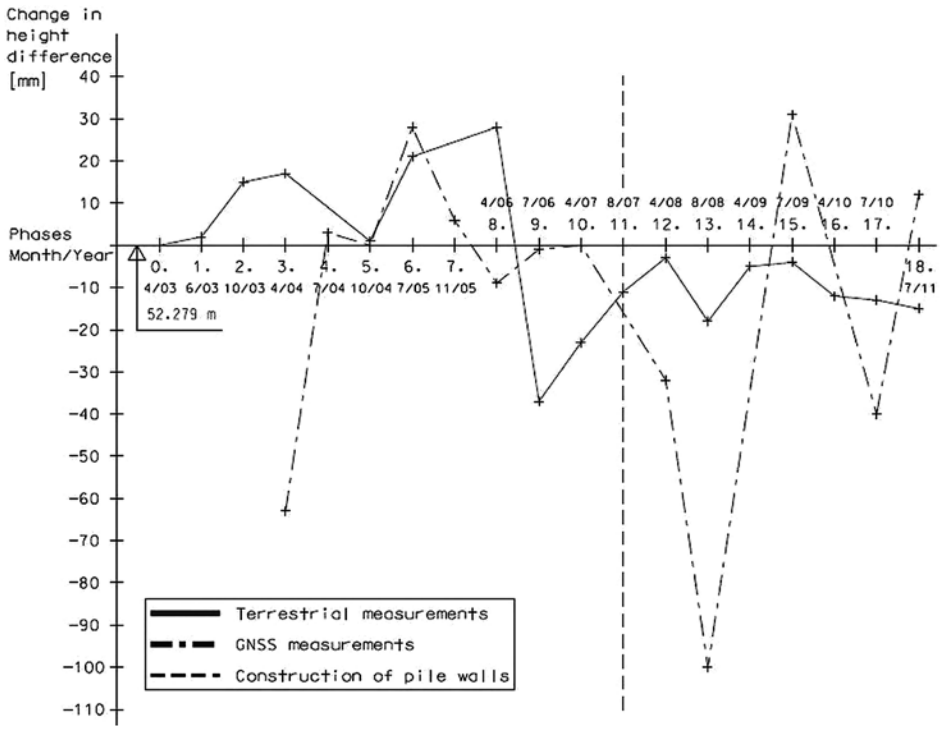


Fig. 9 Changes in height difference between Rab01–Rab02 points. Source: Rytíř 2012, Riegerová 2012

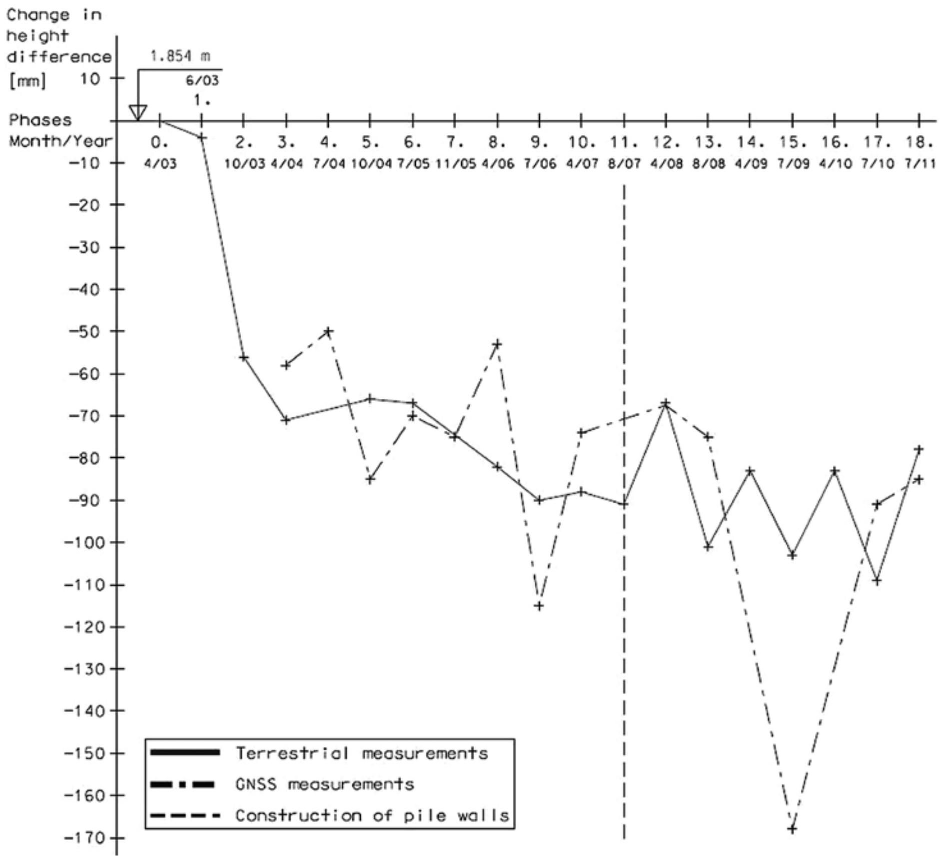


Fig. 10 Changes in height difference between Rab01–Rab03 points. Source: Rytíř 2012, Riegerová 2012

determined as the requirement of the Department of Geotechnics. Points Mpd05–07 are geotechnical monitoring boreholes instrumented with combined casing for the measurement of spatial deformations in the outer dump body and for the determination of the shear plane area and the shear strength of the outer dump (Záleský et al. 2013b). The profile starts at the foot of the slope by the lake surface and ends in the upper part under pile walls (length 830 m, elevation 86 m). The longitudinal profile was measured for each phase after the construction of retaining walls (the last 7 phases) and its objective was to check whether significant spontaneous changes in

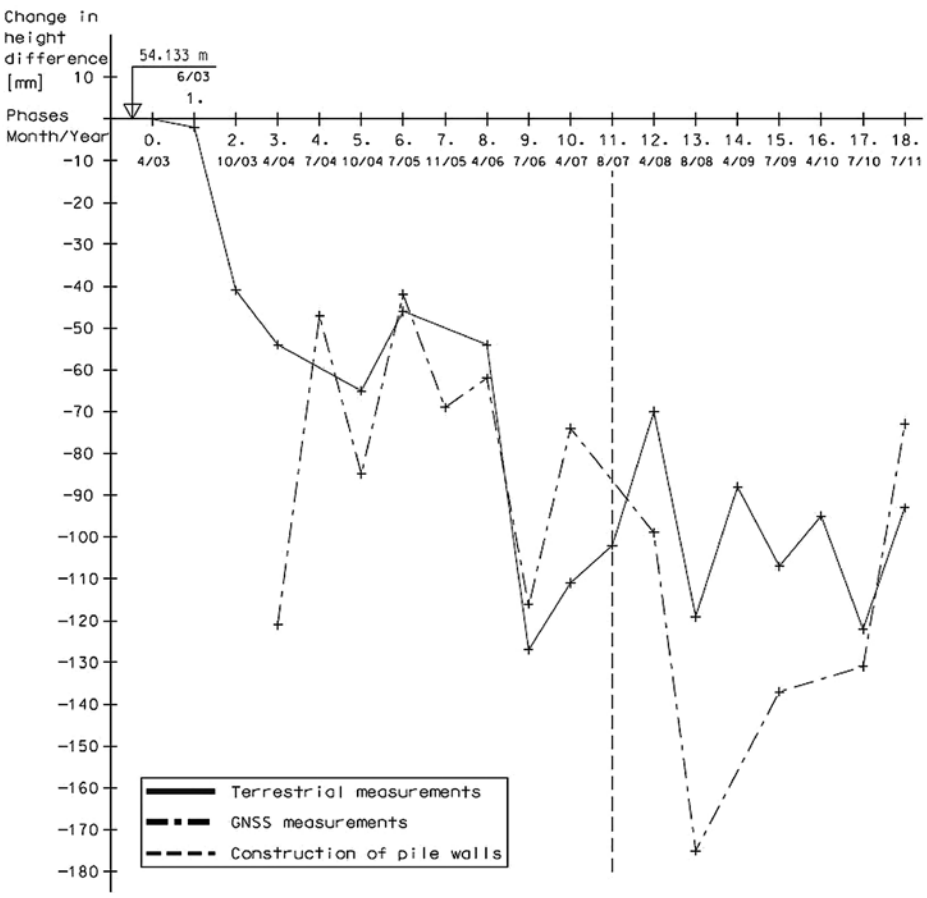


Fig. 11 Changes in height difference between Rab03–Rab02 points. Source: Rytíř 2012, Riegerová 2012

height occur in some parts of the slope (shear plane area determination).

The profile was always surveyed using the standard polar method where the measurement standpoint was selected on the profile axis near point Mpd06. Connecting coordinates in the S-JTSK and Bpv system were taken over from the nearest phases of measurement using the GNSS method. The measured points of the profile were selected on terrain faults (edges of slopes, roads, drainage ditches) and in non-dissected terrain roughly in 25 metre spacing.

The geodetic assessment of individual phases did not reveal any natural changes, only a terrain rise in height by up to 6 m at the foot of the slope during the building of stabilisation benches.

4.5 Digital terrain model

The background data for DTM were obtained by classical terrestrial polar method. Considering the application of laser scanning, the principal negative arguments were the impossibility of placing the scanner on the opposite slope and also the spread of self-seeding vegetation which would dramatically distort the results.

The models are generated in the S-JTSK and Bpv system. Tacheometric measurements were performed in the initial grid of Rab01, Rab02, Rab03 points whose coordinates were determined by the GNSS method in the respective phase (or the nearest previous phase). Detailed survey points in non-dissected terrain were measured in a square grid with sides of 20–40 metres. The main measured elements were artificial embankments and landslides which had to be identified in such a way that they would form closed shapes for the resulting 3D model. Among planimetric elements, paved and unpaved drainage ditches, access roads, a creek bed, concrete culverts, pile walls and geotechnical probes were measured. For the appropriate choice of points, the maximum effort was made to observe the principle of measuring against the slope.

In 2009, the entire territory with the area of 41 ha was surveyed (in phase 14 the southern (A) part and in phase 15 the northern (B) part), and 1366 points were measured for its description (Figure 12). In the following phases, only localities where surface changes occurred were surveyed. In phase 17, the area at the foot of the slope (D) was surveyed, where 3 terrain levels of rockfill had been made between July 2009 and July 2010. Individual places of the terrain had been raised by up to 6 metres and 130,000 m^3 of stone material had been backfilled there. In phase 16 and 18, the southeast (C) part, where continuous landslides and land deformations occurred, was surveyed. Between April 2009 and April 2010, the landslides were levelled, the terrain level was lowered by up to 2 metres in the upper part and 10,000 m^3 of soil were

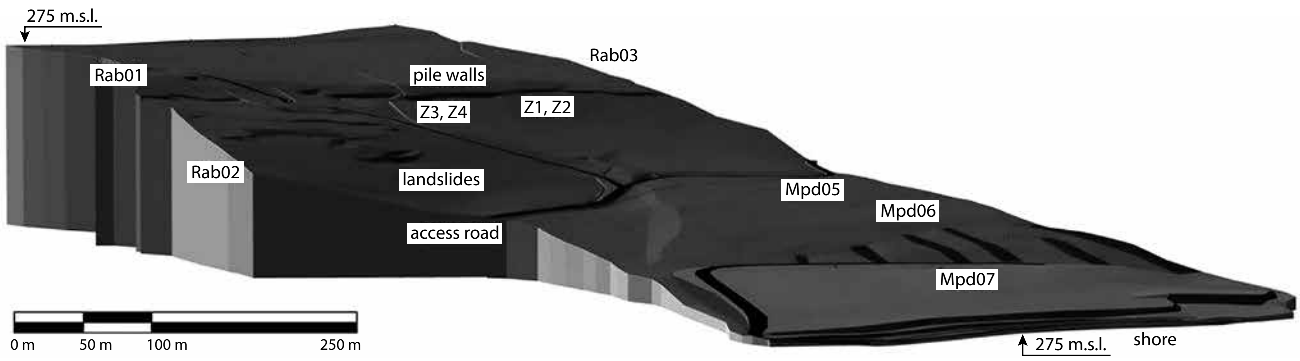


Fig. 12 The slope seen from the northeast (2009).
Source: Braun 2011

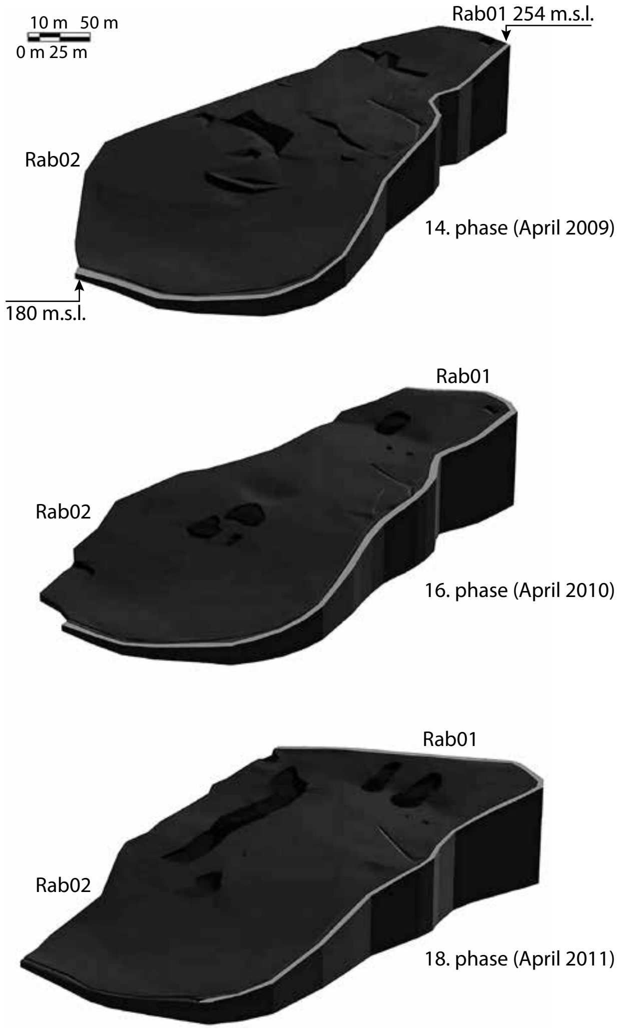


Fig. 13 Comparison of the southwest part – seen from the northwest (04/2009, 04/2010, 07/2011).
Source: Braun 2011, Rytíř 2012

removed. Between April 2010 and July 2011, new landslides appeared on the slope again (Figure 13).

Apart from digital elevation models, contour plans in 1 : 1000 scale were developed for each phase, and the data from DMT are further used by the Department of Geotechnics for the automated generation of longitudinal profiles for research needs.

5. Conclusion

Three basic monitoring and assessment methods were used during the geodetic monitoring of the Rabenov landslide slope, in particular terrestrial measurement in a local surveying grid, GNSS measurement using the fast-static method for the grid connection into the S-JTSK and Bpv national reference systems and detailed measurement using the terrestrial polar method to generate digital terrain models which capture complex changes.

Inclinometric boreholes, 24 meters in depth to ensure the grid stability were used during the local grid establishment; nevertheless, accurate measurements (with the standard deviation of coordinates of ca 2 mm) confirmed the instability of these points, which affected the possibilities of assessing the measurement onto detailed survey points. Based on this experience, potential establishment of larger combined grids with more points stabilised also outside the landslide zone should be considered during the design of detailed survey grids.

The same reference points were not always used during the measurement by GNSS methods, which may have a significant effect on the accuracy of coordinates (the standard deviation in position of ca 14 mm) and the possibility of assessing the shifts of individual points. Because of higher standard deviations in position and in height, this method failed to reliably confirm the stability of the points in the grid. During the construction of new grids, it would be desirable to stabilise more points and to select points outside the monitored zone, but relatively close to it and well protected, as reference points. This would shorten the vectors between reference and rover stations and enhance the accuracy of the assessed coordinates.

The application of detailed tacheometric measurement on small localities of up to 10 ha proved highly efficient in terms of time and a possibility of capturing the major terrain changes which are displayed in a complex digital terrain model. If contact measurement on landslide zones does not pose any risks, this method may be applied arbitrarily, mainly if there are not suitable conditions (vegetation, unsuitable positioning of instruments) for the use of the laser scanner.

Surveys in a local grid manifested that there were continuous movements of observed station points, but the magnitude of changes in position was small depending apparently on the season. On the basis of length changes, persistent positional instability of point Rab02 was presumed (shifts ca 10 mm per year) and by comparing height differences there is an expressed assumption of height instability of point Rab03 (height changes ca 30 mm per season). The comparison of digital models of the southwest part leads to the conclusion that this part was still not fully stable in 2011, and there are mainly hazards of landslides which may destroy the reclamation elements installed there (drainage ditches and roads).

The additional geodetic monitoring would be useful on Rabenov slope. Measurements in grid are unfortunately no longer possible, because point Rab02 was destroyed by reclamation works. Many changes have been on the southwest part and new landslides have appeared on the west side of the pile walls. These facts lead to new DTM, which will be measured in autumn 2013 and spring 2014.

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RÉSUMÉ

Geodetické metody sledování sesuvných území – aplikace na lokalitě Rabenov

Při geodetickém monitoringu sesuvného svahu Rabenov byly použity tři hlavní metody sledování a vyhodnocení, konkrétně terestrické měření v místní geodetické síti, GNSS měření metodou fast-static pro připojení sítě do státních referenčních systémů S-JTSK a Bpv a podrobné měření terestrickou polární metodou pro vytvoření digitálních modelů terénu, které zachycují komplexní změny.

Při zakládání místní sítě byly použity 24 m hluboké inklinometrické vrty, které měly zaručit stabilitu sítě, ovšem přesným měřením (směrodatná odchylka souřadnic cca 2 mm) byla potvrzena nestálost těchto bodů, což ovlivnilo možnosti vyhodnocení měření na podrobné pozorované body.

Při měření metodami GNSS nebyly vždy použity stejné referenční body, což může mít významný vliv na přesnost souřadnic (směrodatná odchylka v poloze cca 14 mm) a možnost vyhodnocení posunů jednotlivých bodů. Kvůli vyšším směrodatným odchylkám v poloze a ve výšce nebyla touto metodou spolehlivě potvrzena stálost bodů sítě.

Při budování nových sítí by bylo vhodné stabilizovat více bodů a vytvořit tak komplexnější síť i s body mimo sledované oblasti, které by sloužily jako referenční pro GNSS i terestrické měření.

Použití etapového podrobného tachymetrického měření na malých lokalitách do 10 ha se ukázalo velmi efektivní z hlediska času i možnosti zachytit nejdůležitější změny terénu, které se zobrazují v komplexním digitálním modelu terénu.

Geodetické měření v místní síti prokázalo, že stále dochází k pohybům stanoviskových pozorovaných bodů, ale velikost změn polohy je malá a zřejmě závislá i na ročním období (posun bodu Rab02 v poloze o 10 mm za rok, výškové změny o velikosti až 30 mm bodu Rab03 za roční období). Z porovnání digitálních modelů jihozápadní části lze usoudit, že tato část v roce 2011 ještě nebyla plně stabilní a hrozí v ní zejména svahové zátrhy, které mohou zničit vybudované rekultivační prvky (odvodňovací příkopy a cesty).

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