DETERMINING THE POSITION OF POINTS ON GEOLOGICALLY UNSTABLE LAND WITH GEODETIC MONITORING

DOLOČANJE POLOŽAJA TOČK NA GEOLOŠKO NESTABILNIH TLEH Z GEODETSKIM MONITORINGOM

Boštjan Kovačič
University of Maribor,
Faculty of civil engineering, transportation engineering and architecture
Smetanova 17, 2000 Maribor, Slovenia
E-mail: bostjan.kovacic@um.si

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Keywords

geodesy, geotechnics, engineering, monitoring, GNSS measurements

Abstract

An accurate determination of tectonic displacements is very important for the safety of people and animals as well as for natural and artificial objects. When establishing the short-term and long-term displacements of land or objects, the expected displacements should be anticipated in order to define the method of geodetic monitoring, which has become a regular feature in geotechnical monitoring.
In general, the land growling is determined by geodetic methods: the trigonometric method predominates in short--term monitoring, whereas the GNSS (Global Navigation Satellite System) method is mostly used for long-term monitoring, depending on the properties, complexity and duration of each project. Both methods provide reliable results with accuracy range of a few millimetres. This article focuses on the GNSS method, which has become irreplaceable for the monitoring of land growling. The article describes a procedure for determining the process in the case study of Razdrto-Vipava highway, where displacements have been monitored since 2007. Moreover, the process of contacting the European control point and procedure for levelling the measured values are explained.

Ključne besede

geodezija, geotehnika, inženirstvo, monitoring, GNSS meritve

Izvleček

Natančno ugotavljanje tektonskih pomikov je zelo pomembno za varnost ljudi, živali ter zgrajenih in naravnih objektov. Kadar ugotavljamo kratkoročne ali dolgoročne pomike moramo poznati pričakovane pomike, saj se na osnovi tega odločimo za metodo geodetskega monitoringa, ki je postala stalnica pri geotehničnih monitoringih. Z geodetskimi metodami največkrat ugotavljamo plazenje tal. Pri kratkoročnih monitoringih izstopa trigonometrična metoda medtem, ko se GNSS (Global Navigation Satellite System) metoda večinoma uporablja za dolgoročne monitoringe, ki so odvisni od lastnosti, kompleksnosti in trajanja projekta. Obe metodi nam dajeta zanesljive rezultate v območju natačnosti nekaj milimetrov. V prispevku se bomo osredotočili na metodo GNSS, ki je postala nenadomestljiva metoda pri spremljanju plazenja tal. Opisan je postopek določitve pomikov na primeru HC Razdrto-Vipava, kjer pomike spremljamo od leta 2007. Opisan je postopek navezave na evropske kontrolne točke in postopek izravnave izmerjenih vrednosti.

1 INTRODUCTION

The structures that are usually the main focus in deformation monitoring are often only an indirect indicator of the dynamic processes associated with land surface and undersurfaces. Today, artificial structures are built from materials and using methods to mitigate the effects of the deformation of natural objects in terms of their generation. Therefore, one of the primary tasks prior to construction is to determine the stability of the surface, which is related to the level of rock sliding or crawling under the surface as well as to the geotectonic movements deeper inside the earth. Thus, geological and geotectonic investigations, which often replace deformation monitoring and are focused exclusively on surface changes, are undertaken as urgent works before the construction of geotechnical facilities begins. These investigations usually start after the beginning, rather than before, or even after the completion of the structure's construction. As a rule, the reasons for this inconsistent sequence of events are economic. Crawling usually reveals itself through errors in the inclinations or the orientation of the investigated object that can be: new changes that do not show any signs of unexpected tilt in the past, changes in the currently active landslide, or the consequence of the proximity of the areas with the existing geodynamic events. The aim of deformation monitoring is to obtain the facts about the stability of the observed object or surface, expressed by the size of the movements of certain points. In addition to stability, the importance of determining movements lies in evaluating the potential hazard of built and natural objects for the environment and above all for human life and property. Therefore, the determination of movements and the deformations of natural and artificial structures are among the most engaging and most demanding tasks in geodesy. Deformation monitoring covers several temporal segments in terms of determining movements [1] at the time of the occurrence of natural forces leading to conditions for the emergence of the movement, at the time of the operation of natural forces or during the construction operation, triggering the realization of the movement of natural or man-made structures, at the time of the settling of natural forces or after the completion of the construction, establishing a long-term trend of movement of objects in a given area, at the time of the cessation of activity or natural forces or during the steady influence of natural forces on the buildings, resulting in the relative elimination of the movements or the occurrence of stagnation of the changes of these movements. So, deformation monitoring covers a number of spatial segments in terms of determining the movement: at the site of the origin of the phenomena of the operation of natural forces, at the site of the effective functioning of the natural forces or in the immediate vicinity of the facility, at all detectable sites of the functioning of natural forces, in places, where according to the natural forces the influence is relatively stable, and indirectly also in places where according to the operation of natural forces, influences are absolutely stable. Given the fact that there are no absolutely stable points, especially on the earth's surface, the determination of displacements and deformations is presently practical in all scientific disciplines that are in one way or another related to the physical environment. The position for determining the land slide is defined by the geomechanics in cooperation with the surveyor who assesses the suitability of the position due to the purity of the signal for the purposes of satellite deformation monitoring. Global Navigation Satellite Systems (GNSSs) with Carrier Phase (CP) based on high-precision positioning techniques have been widely used in geodesy, altitude determination, engineering survey, agricultural applications, cataster mapping, etc. Considering the modernization of GNSS, multi-constellation and multi-frequency data processing is one of the focuses of current GNSS research. The GNSS development authorities have developed better designs for the new signals, which are aimed at fast acquisition for civil users, are less susceptible to interference and multipath and have a lower measurement noise. However, how good are the new signals in practice? The signal quality of the multi-GNSS (GPS, GLONASS, Galileo, BDS and QZSS) is assessed by looking at their zero-baseline Double Difference (DD) CP residuals. The impacts of multi-GNSS multi-frequency signals on single-epoch positioning are investigated in terms of accuracy, precision and fixed solution availability with known short baselines [2]. Today, GNSSs are used for a multitude of applications around the world, and there is a general quest for better positioning accuracy and reliability, as well as faster position acquisition from both user groups and the GNSS research community. Combining observations from multiple GNSSs in one positioning process and/or using multiple frequencies from one or more GNSSs is an important step towards reaching these goals. Accounting for all error sources in the positioning process, including hardware biases, is a prerequisite for accurate results [3].

2 MEASUREMENT METHODS

On the basis of previous research we decided to analyse our measurements in detail, which were performed with the GNSS method. The detailed measurements were performed using movements of the Slovenian GNSS reference network named SIGNAL and the local reference points in the investigated area named BAZ1 and BAZ2. The network SIGNAL (Slovenia-Geodesy-Navigation-Location) is a Slovenian network of permanent GNSS-stations and is a basis for the country's geographic information infrastructure. It includes 16 stations that are evenly distributed across the country, as shown in Figure 1.

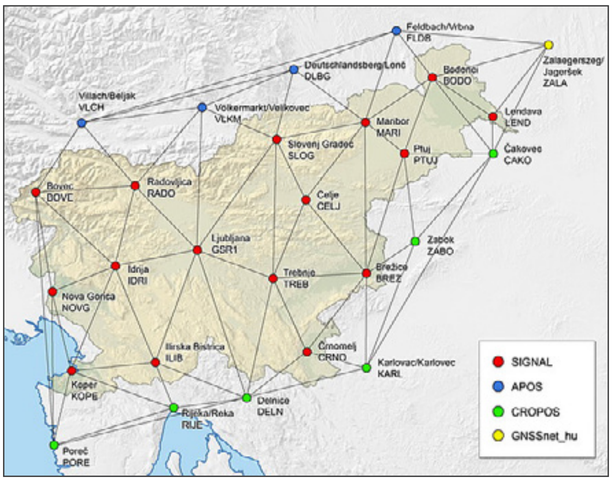


Figure 1. References point of network SIGNAL [4].

The GNSS station network is important for users when they want to determine the relative or differential position of their own receivers according to the chosen GNSS station in the network. Such a determination of the position is incomparably more accurate than the absolute position, established without connection to the network. In order to connect GNSS measurements with the physical realisation of the coordinate system, two GNSS receivers are needed. Obviously, the network allows for more rational measurements, because it replaces the users' reference receivers, so they only need a movable receiver, while the role of the reference receiver is taken by the SIGNAL network system. The Ljubljana permanent station is also included in the European Permanent Network, the data from this station are regularly sent to the data centre EUREF (European Reference Frame), which operates in the frame of the International Association of Geodesy – IAG. Two-frequency GNSS receivers with antennas are placed on all network stations and the continuous monitoring is performed 24 hours a day and 365 days a year.

The chosen station location allows for a fairly even distribution across the country, with maximum distances between the stations that are less than 70 km. This distance between permanent GNSS stations enables users to achieve high accuracy of the position determination in the whole area of the country if they use VRS observations. The calculation of VRS observations is possible only in the networks that are centrally controlled. In fact, GNNS service controls and manages the SIGNAL network from the network centre. This uses a programme package for the central network control (Trimble Pivot Platform) to collect and process the observations from all the GNSS stations in the network that flow into the centre in real time. The programme package enables the continuous calculation of corrections in the ionosphere, troposphere, satellite positions and an unknown number of ambiguities for each station in the network and the calculation of VRS observations for a random location in the network.

The user's receiver (BAZ1 and BAZ2) must enable two-way communication, i.e., data transmission and reception. After the determination of the absolute position, this is sent in the form of a NMEA message into the network centre. Based on observations of all the GNSS stations, the model of impacts on observation is generated in the centre, then the interpolated values of the impacts are calculated for the user's position. Furthermore, the values of the observations, performed by a physical receiver on the location near the user, are calculated regarding the modelled influence. These observations are called VRS observations; the position for which they are generated is named the Virtual Reference Station (VRS). The VRS data are then sent from the network centre in the form of an RTCM message; they are dealt with by the user in the same way as if they were RTCM messages observed by a GNSS station. The user's manner of position determination stays the same; despite the long distance from the nearest GNSS station, the accuracy of the position determination is very high [4].

2.1 Methods of Precision Geodetic Survey Techniques

Generally, specifications for the precision of geodetic survey techniques include the least angular count of instruments to be used, the number of observations, the rejection criteria of the observations, the spacing of major stations and the expected angular and positional tolerances. To obtain exact measurements, the surveyor must use precise equipment and precise techniques. Many of the techniques used in precise surveys are adapted from the conventional geodetic positioning methods and instrumentation, but with some differences, such as survey procedures. In addition, conventional horizontal and vertical survey techniques use traditional ground survey instruments (electromagnetic distance measurement, robotic total stations, levelling) and the GNSS survey techniques [5]. The spatial geodetic monitoring of civil constructions is one of the most complicated tasks of engineering geodesy. Modern survey equipment allows for the simplification of many technological processes. At the same time, new equipment is leading to the invention of advanced spatial monitoring methods. Nowadays, accurate and precise total stations with automatic target recognition (ATR) and reflector less modes have become relatively common [6], [7]. Exceptionally popular are the motorized total stations [8]. In engineering practice, the deformation monitoring of large structures (bridges, viaducts, landscape, hydro power plants, etc.) uses various geodetic methods, such as levelling, trigonometric heights and GNSS Recently, the state geodynamics network is, more or less, observed by the application of these technologies. Although the GNSS allows positioning with accuracy comparable to conventional methods, the use in the past was limited due to the long duration of the observation. The development of instruments and in particular software, have significantly reduced the time of observation of the GNSS method so that they can, in some cases, ensure 1-mm accuracy in less than 1 hour of observation. However, due to the elimination of the global movement of ground water and land masses, it is recommended to increase the length of the measurement time, which should last at least 24 hours, continuously. Determining the position with GNSS depends on the types of observed quantities and the method of data processing: absolute, differential code and the relative phase-shift method. In addition to the classic kinematics and RTK methods, the modern approach also includes the VRS method (Virtual Reference Station) and the PPP (Precise Point Positioning) method. The latter is a more accurate method of determining the absolute position that can be carried out as static or kinematic. For the purposes of the geodetic monitoring, only the static mode with millimetres accuracy applies explicitly [9]. Research in recent years presents the use of GNSS for a precise determination of 3D positions in the control of hazardous natural phenomena. Detailed analysis of the landslide movements, especially for the needs of the security system installation, functioning in real time, requires a combination of accurate positioning in three dimensions (a few mm) and a high temporal resolution (less than 1 hour). Monitoring of landslides with GNSS technology is typically used in several epochs, measured as a complement to conventional methods. It is necessary to determine the time period of the observations to obtain a quality GNSS monitoring method, which is the only way to get the precision around 1 millimetre. Otherwise, all the described GNSS methods based on the same geometric principle, namely, that the position of a point in space is determined using the crossings of the spheres whose radius represents the distance measured from the new point to more distant points. These points in the GNSS system materialize themselves as satellites around the Earth. The principle is similar, almost the same as in terrestrial trilateration: according to the law of planar geometry, the position of a new point is determined by the intersection of the circular lines, where the values of the radii are determined by the measured distances, the centre of circles, however, determines the standing positions, from which distances are measured. In principle, three measured distances (receiver Satellite) would be enough, but it is required, for the movement of the satellites and the Earth and the difficulties in determining the exact status of clocks at the time of submission and acceptance of the signals, to obtain observations of at least four satellites. Namely, the determination of the exact signals receiving time demands an extremely accurate clock in the receiver. These requirements can be reduced by using the time signal of a fourth satellite, therefore, in such case only the time differences of the receptions of signals from various satellites can be measured. The method of GNSS in the deformation monitoring is a relatively new method, which brings enormous advantages versus the conventional method. Precision positioning and determination of the relative coordinates of the points by this method largely depends on the deployment of the satellites at the time of the quality and performance measurement of the observations (from the precision of the field measurements, the consistency of the data, from the observance of the rules for locating points for the duration measurements, etc.). The accuracy can be better described through the evaluation of the effects that cause measurement errors. The main sources of errors that contaminate the information obtained from GNSS technology can be divided into three groups [10]:

a) The spread of the signal error, tropospheric and ionospheric refraction, multipath.
b) Error in connection with the receiver: an error of determination of the antenna phase centre, the system noise of the receiver, irremovable effects of the multipath, incorrect coordinates of the standing points, which are in the process of processing proclaimed as given.
c) Error in connection with the satellites: the determination of the error of positions and tracks of satellites.

Of course, this applies for a 24-hour observation. Based on the code or phase of GNSS observations, the absolute and relative position of the GNSS receiver can be obtained. Absolute position is determined solely on the basis of the positions of GNSS satellites in the chosen coordinate system at the time of the observations and the observed distances between the satellite and the receiver. The relative position is fixed relative to the known position of one or more points placed in the default coordinate system at the given positions of the satellites and the observed distance between the satellite and the receiver. In both cases, therefore, the basis for position determining is a measurement of the geometric distance between the satellite and the receiver.

3. GNSS MEASUREMENTS OF DISPLACEMENTS IN A SLOVENIAN MOTORWAY SECTION UNDER CONSTRUCTION

Prior to the observations, 50 points were permanently stabilized on the route. Half of them were positioned on the structures, while the rest were in the vicinity in the triangular network. Concrete pillars were used for the stabilization of plates installed on the objects. Since this is a very demanding configuration of the terrain, the measuring points were positioned in such a way that two points were outside the deformation area (BAZ1 and BAZ2), which also represents the base for the adjustment (Fig2a). Based on geomechanics research, two local reference points, BAZ1 and BAZ2, are placed in the area of the ground-displacement measurements and represent the beginning and the end of the observed area or base vector. These two points are reference points of our local GNSS network and are connected to four stations of the SIGNAL permanent network. According to the SIGNAL points, the displacement of the base vector BAZ1–BAZ2 is determined. In the process of calculating the other points on the motorway route, first these two points are levelled. All other observations at 50 points are then calculated according to these two base points. Principally, the base points should be connected to the four nearest GNSS points of the SIGNAL network that is included in the European GNSS network (IGS), which provides the reductions of displacement of the Eurasian Plate and transformation parameters so that the coordinates can be converted into the coordinate system (ETRS). It is important to choose those points from the SIGNAL network that are approximately equally far from the site and correctly distributed around it.

Other points were mostly placed on the landslide area. The changes in the position of the points in the deformation area are compared based on the base line. Furthermore, these points can serve to carry out classical geodetic surveying of each structure's movements. Eight Topcon HypePro, 1 Leica and 2 base CR3 Topon GNSS antennas were used (Fig 2b). In order to obtain better results according to the satellite positions, all the points were continuously measured for 24 hours. In that case, data capturing took place for 6 days for all the points. It is important to place exactly the same antenna at the same point in each epoch, because of the different phase centres of the different antennas. Consequently, some possible antenna errors can be avoided in this way, such as the error of the antenna centre, the different antenna centre heights, different frequencies, the antenna sensitivity to multipath, the sensitivity to deviation of a vertical line, etc.

Figure 2a. Base point BAZ1 and BAZ2.

Figure 2b. GNSS equipment.

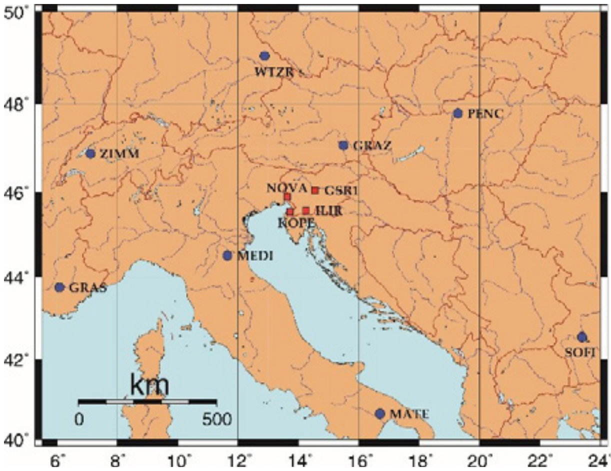


Figure 3. Reference points in Slovenia and orientation points [11].

First, it was necessary to define a geodetic datum that was provided by a set of given IGS points (International GNSS Service) with given coordinates and velocity vectors in the International Terrestrial Reference Frame 2005 with high accuracy. In addition, four GNSS points (KOPE, ILIR, NOVA, GSR1) of the Slovenian network of permanent GNSS stations SIGNAL were also considered in the processing, but as new points. The SIGNAL network points have lower accuracy, so that they cannot be treated as given. In Fig 3, the reference points in Slovenia are in red, the points in the surrounding area that were used for orientation and adjustment are in blue.

The digital orthophoto (Fig 4) shows the disposition of some points for observations.

The data in the antenna are stored according to the Julian calendar. The processing of observations is made separately for each for each Julian date (for example, 16 September 2018 is according to Julian calendar 2458391). For this reason, two sets of coordinates have to be obtained for each point. Based on the differences or the similarities of these two positions, the quality of the observations and the processing of these can be considered. The result of the processing is the evaluation of the coordinates of the points for each day with the respective accuracy. After the processing of observations, one set of coordinates in the ITRF2005 coordinate system is obtained for different time epoch for each point. The goal is to present the coordinates of points

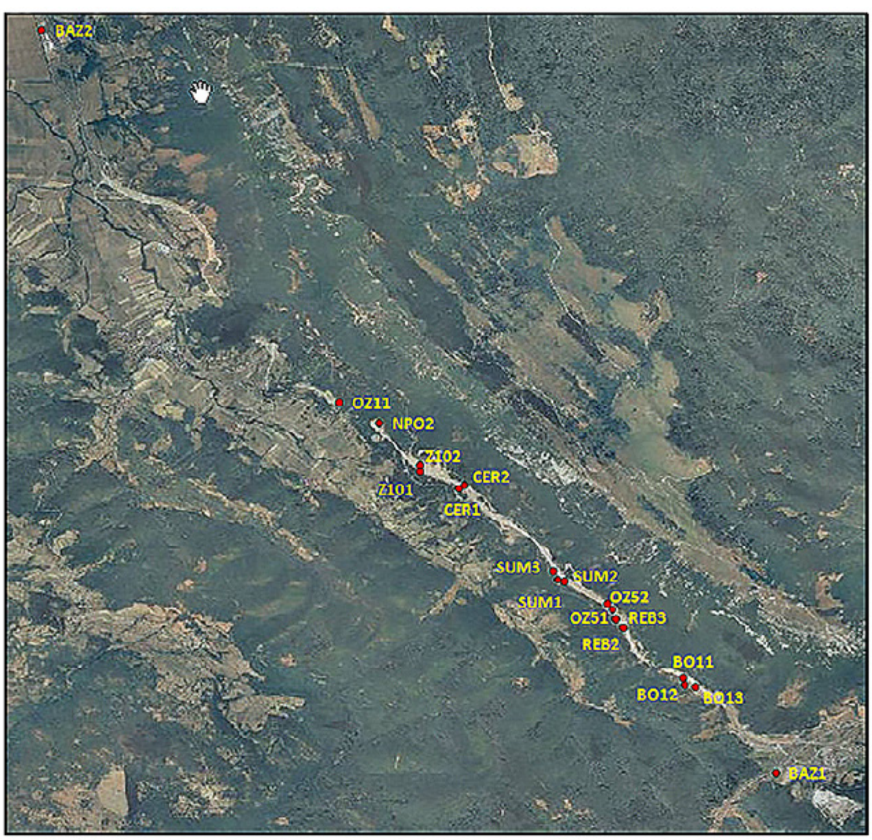


Figure 4. Digital orthophoto and location of observation points [11].

in the national projection, i.e., the transverse Mercator projection with its associated accuracy. The transition from the ITRF2005 to the state projection is carried out through the transition to the ETRF89 coordinate frame in which the positions (according to ITRF2005) are reduced for the movement of the Eurasian tectonic plate. The point's positions in ETRF89 are the basis for conversion into a national cartographic projection. The accuracy of the point coordinates in the cartographic projection assumes the same accuracy as the geodetic coordinates in the ITRF2005. In addition, it is assumed that the transition to the state cartographic projection (through ETRF89) does not change the precision of the coordinates significantly [11].

Table 1 presents the coordinates of the points in the transverse Mercator projection with the corresponding accuracy. The altitudes are obtained by decreasing the ellipsoid height, estimated through the observations of the GNSS for the geoid altitudes obtained from the current geoid model of Slovenia, as shown in Fig 5.

The coordinates of the points and their accuracy in the individual measurements are determined as superposition of two systems of normal equations. From Table 1 and Table 2 it can be seen that the data with a positional accuracy of 1 to 2 mm were obtained. For the high components, the accuracy is slightly worse due to the poor reception of the signal and the multi path. This was compensated with classic terrestrial geodetic measurements.

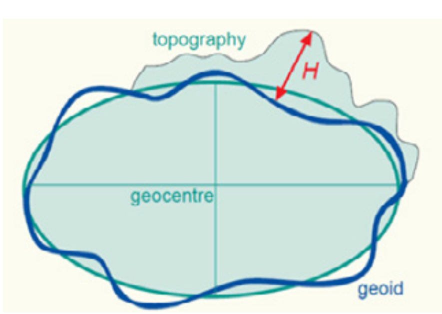
Tables 3 and 4 show calculated values of the displacement for base points 1 and 2 in the period from November 2007 to October 2014 according to single periods. The red field displays the common displacement for this period. The results show that the common position displacement of the BAZ1 point is 2.8 mm and BAZ2 is 22 mm. The reason for the displacement to BAZ2 seems to be the irregular position of a column and its poor installation, since it is placed next to the parking lot of road maintenance vehicles, which is subject to deformation of the land because of the weight. Fig 6 and 7 show a vector display of the BAZ1 2and BAZ2 point displacements in each period. The red vector presents the common displacement from the beginning of the monitoring. The displacement of 0.0126m was detected at the BAZ1 point by measurements in 2008. The results from this year show very dynamic displacements of the BAZ1 and two points that can be attributed to the poorer arrangement of the satellites in the time of measurements and shorter time of data gathering;

Table 1. Measured values and accuracy of measurements at the reference points in October 2008.

Point	Y [m]	X [m]	H [m]	σ_Y [mm]	σ_X [mm]	σ_H [mm]
BAZ1	426090,0469	68720,5530	627,0367	1,85	0,87	1,93
BAZ2	418920,5057	77178,8702	112,9227	1,29	0,67	5,33

Table 2. Measured values and accuracy of measurements at the reference points in October 2016.

Point	Y [m]	X [m]	H [m]	σ_Y [mm]	σ_X [mm]	σ_H [mm]
BAZ1	426090,0541	68720,5410	627,0282	1,13	1,01	1,32
BAZ2	418920,5132	77178,8854	112,8537	1,41	0,51	3,97



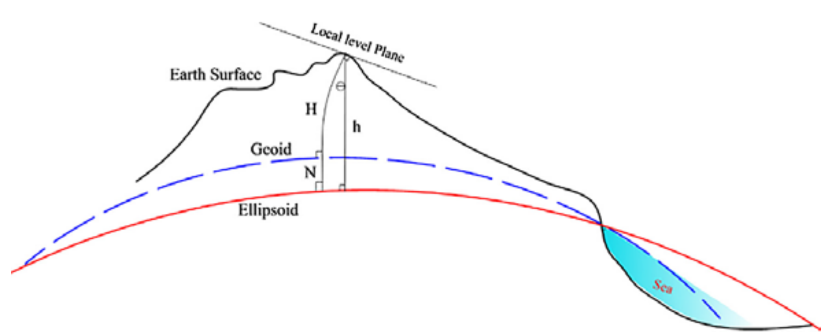


Figure 5. Altitudes on Geoid and reference ellipsoid (www.google.si/geoid).

Table 3. Calculated values for displacement of BAZ1 point [12].

	Y [m]	X [m]	Z [m]	d_y [m]	d_x [m]	σ_y [m]	σ_x [m]	Displacement 2D [m]	date
BAZ1	426462.018	68234.244	627.039	-	-	-	-	-	2007_11
	426462.0253	68234.2393	627.0248	0.0073	-0.0047	-	-	0.0087	2008_1
	426462.0197	68234.2423	627.0297	0.0017	-0.0017	-	-	0.0024	2008_2
	426462.0306	68234.2432	627.0350	0.0126	-0.0008	-	-	0.0126	2008_4
	426462.0231	68234.2410	627.0264	0.0051	-0.0030	-	-	0.0059	2008_10
	426462.0163	68234.2401	627.0299	-0.0017	-0.0039	-	-	0.0043	2009_3
	426462.0233	68234.2420	627.0254	0.0053	-0.0020	-	-	0.0057	2009_5
	426462.0240	68234.2403	627.0349	0.0060	-0.0037	-	-	0.0070	2010_2
	426462.0257	68234.2441	627.0102	0.0077	0.0001	-	-	0.0077	2010_10
	426462.0157	68234.2453	627.0407	-0.0023	0.0013	-	-	0.0026	2013_6
	426462.0210	68234.2415	627.0066	0.0030	-0.0025	-	-	0.0039	2013_10
	426462.0195	68234.2464	627.0222	0.0015	0.0024	0.0014	0.0016	0.0028	2014_10

Table 4. Calculated values for the displacement of the BAZ2 point [12].

	Y [m]	X [m]	Z [m]	d_y [m]	d_x [m]	σ_y [m]	σ_x [m]	Displacement 2D [m]	date
BAZ1	419292.449	76692.184	112.924	-	-	-	-	-	2007_11
	419292.4539	76692.1837	112.9186	0.0049	-0.0003	-	-	0.0049	2008_1
	419292.4610	76692.1860	112.9060	0.0120	0.0020	-	-	0.0122	2008_2
	419292.4496	76692.1816	112.9101	0.0006	-0.0024	-	-	0.0025	2008_10
	419292.4511	76692.1817	112.9185	0.0021	-0.0023	-	-	0.0031	2009_3
	419292.4505	76692.1845	112.9035	0.0015	0.0005	-	-	0.0016	2009_5
	419292.4465	76692.1848	112.9172	-0.0025	0.0008	-	-	0.0026	2010_2
	419292.4495	76692.1847	112.9234	0.0005	0.0007	-	-	0.0009	2010_10
	419292.4325	76692.1866	112.9143	-0.0165	0.0026	-	-	0.0167	2013_6
	419292.4312	76692.1844	112.8840	-0.0178	0.0004	-	-	0.0178	2013_10
	419292.4270	76692.1858	112.8912	-0.0220	0.0018	0.0015	0.0017	0.0221	2014_10

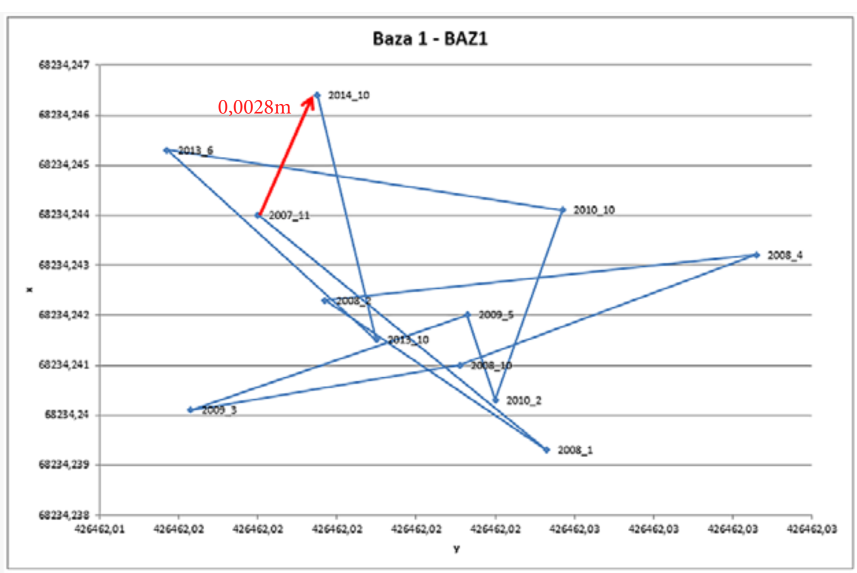


Figure 6. Vector display for the displacement of the BAZ1 point [12].

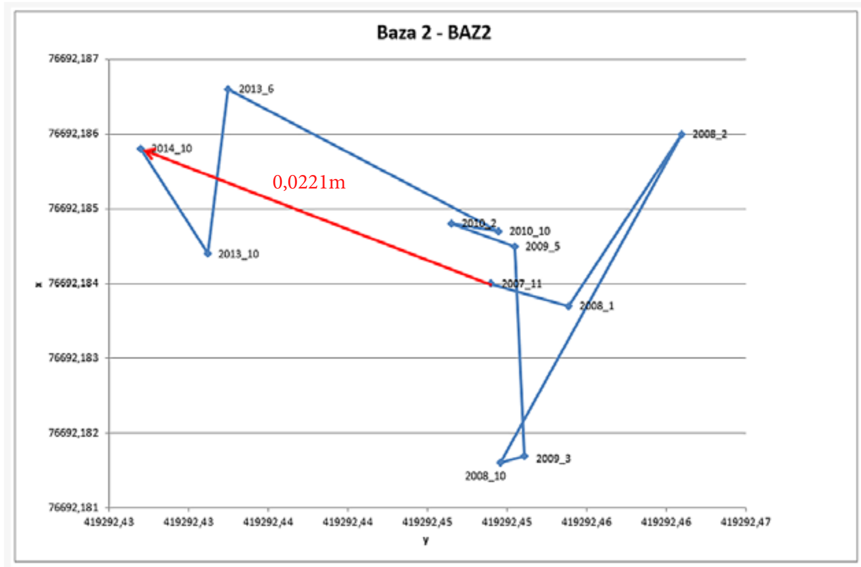


Figure 7. Vector display for the displacement of the BAZ2 point [12].

therefore four revisions were carried out this year. In fact, the displacement of the base points does not influence the results of the other observed points, because the displacement of the base points is levelled. The base points represent the base vector, from which the relative displacements of the observed points are calculated and they serve as control points. Two points of the motorway section, BAZ1 and BAZ2, were established in order to eventually define the coordinate base of the motorway section. The results of the data processing show that the coordinates of both points changed significantly. Furthermore, the transformation from the January and February 2008 measurements to the November 2007 results in coordinates of all measured points, where the coordinate base is determined by the reference points BAZ1 and BAZ2.

In principal, any assessment of possible movements of geodetic points obtained from GNSS measurements requires a determination of the coordinates for points in several time epochs. It is necessary to estimate the trend of possible coordinate changes based on a series of obtained coordinates. The trend of changes in the coordinate points can be well defined only in the case of high-quality coordinate points or by quality time series of coordinate points. Furthermore, the displacement results are only shown for one of the reference points of the SIGNAL network, named KOPE, which is obviously also included into European GNSS network (IGS). Displacements of the reference points should be taken into account when measuring base points, from which the displacements of the points are calculated within the base vector. As a rule, relative displacements in the relation base – vector – object are established. The points of the SIGNAL network are very important in this particular case because the displacement of the fundamental base vector is determined from them.

Fig 8 shows the changes in the coordinate points in time for a 1-year period (2007/08) for the KOPE reference point in the form of deviations from the mean for each coordinate of each point in the network. The coordinate in each epoch represents a deviation from this mean with the given standard deviation.

Furthermore, changes in the point coordinates can be displayed in the plane of the cartographic projection. Fig 7 shows the estimated point coordinates for each time epoch. However, on the basis of the calculated standard deviations of the estimated point coordinates and the method used for measurements, it would be difficult to reliably determine the size of the movements of the points. The movements at the other reference points of the SIGNAL network are shown in Fig 9.

4 DISCUSSION

Today, space overcrowding results in man-made structures erected on areas where we could not imagine buildings years ago. We need to be aware that new technologies enable us to build on the terrain that is very unpredictable, therefore monitoring is absolutely needed. In Slovenia, one highway section

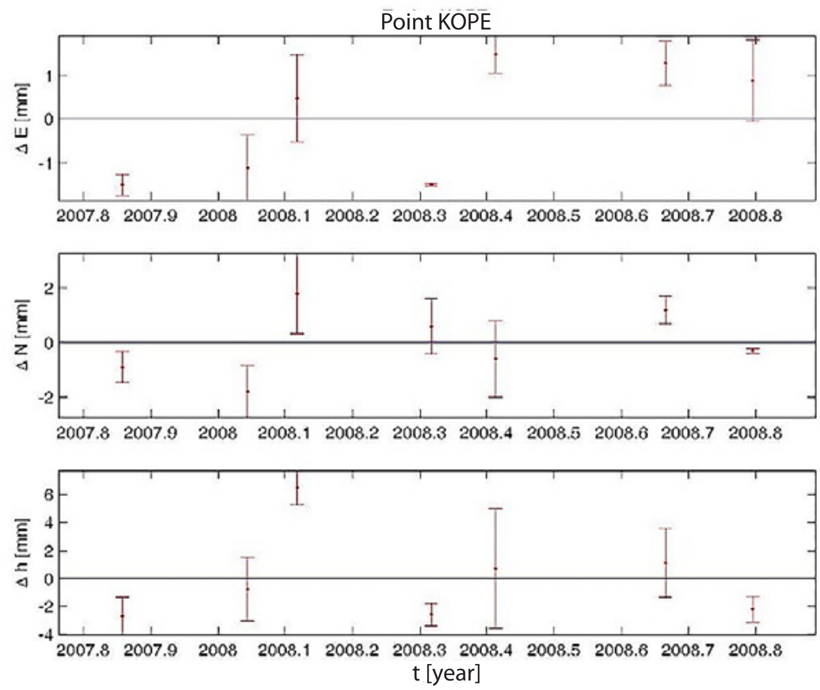


Figure 8. Changes in the coordinate points for a 1-year period [11].

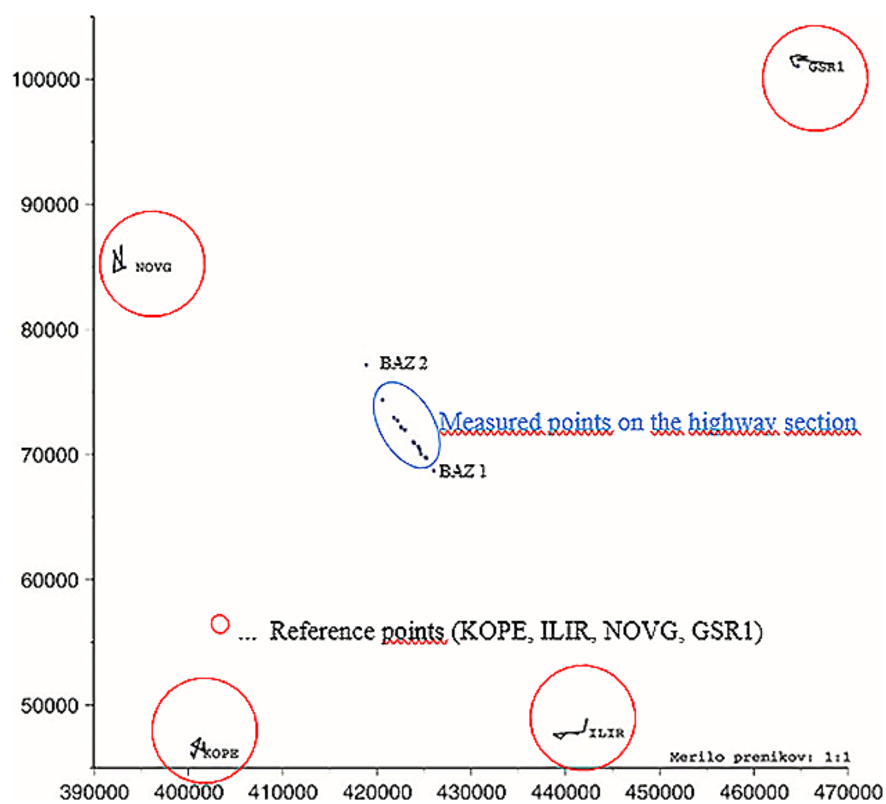


Figure 9. Movements of the reference points of the SIGNAL network [11].

was constructed on the ground, where the terrain was expected to slide under the weight of the objects, but only to the predicted values. Unfortunately, these values quickly reached their maximum and exceeded the permitted limits. In order to determine whether the terrain and objects are sliding according to the predictions, monitoring must be carried out. This is most often done using geodetic instruments such as GNSS. First, it is necessary to check the stability of the reference points from which we perform a variety of geodetic measurements. The position of the reference points is obtained by means of a deformation analysis. The main goal of the deformation analysis is to confirm the stability of the reference points, resulting in the determination of movements of the control points in the geodetic network, which is developed in the vicinity of the observed objects. We have described the process of data acquisition and the linearization of the distance between the satellite and the receiver. It should be recognized that geodetic monitoring is a very demanding and complex task, which must be carried out precisely and reliably.

5 CONCLUSION

A deformation analysis can be described as a set of methods for the detection and evaluation of movements or the deformations of natural or artificial objects. Given that attention is mainly focused on the processing of data, a deformation analysis often denotes only the process of determining the movements using the relevant analytical approaches or indicates the method of processing the measurement results. Geodetic deformation monitoring is therefore a wider concept, covering all the phases from planning and system set-up up to continuous operation, data processing, analysis and presentation of results, while the focus under deformation analysis is the processing of the measured data and their analysis. The deformation analysis of GNSS data for the monitoring of soil movements should also be used in the field of statistics, geomechanics, civil engineering and geology and as additional knowledge in the field of geophysics, geodynamics and surveying.

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