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THE INFLUENCE OF GEOTECHNICAL CONDITIONS ON THE INSTABILITY OF ROAD EMBANKMENTS AND METHODS OF PROTECTING THEM

WPŁYW WARUNKÓW GEOTECHNICZNYCH NA NIESTATECZNOŚĆ NASYPÓW DROGOWYCH I SPOSOBY ICH ZABEZPIECZENIA

A b s t r a c t

This article discusses the problem of the instability of road embankments. Two types of landslides located in various geotechnical conditions were analysed. The first case is where the stability of the road embankment itself is lost, in which the soil layers under the embankment have no influence. In the other case, the instability of the embankment is connected with landslides of the soil on which a given embankment is situated (slope stability loss). The authors proposed original solutions which were later on verified by MIDAS GTS NX®. The conducted studies show that the proposed protection strategies for both slopes are effective, thus yielding a high coefficient of general stability (FoS).

Keywords: instability, road embankment, landslide, flysch Carpathians, slope, Factor of Safety

S t r e s z c z e n i e

W artykule został poruszony problem niestateczności nasypów w drogownictwie. Analizie zostały poddane dwa rodzaje osuwisk drogowych zlokalizowanych w różnych warunkach geotechnicznych. W pierwszym przypadku doszło do utraty stateczności samego nasypu drogowego, na którą wpływu nie mają warstwy gruntu zalegające pod nasypem. W drugim przypadku niestateczność nasypu komunikacyjnego związana jest z osuwaniem się gruntów, na którym przedmiotowy nasyp się znajduje (utrata stateczności zbocza). Zostały zaproponowane autorskie rozwiązania, które następnie zweryfikowano w programie MIDAS GTS NX®. Przeprowadzone badania wykazały, że projektowane zabezpieczenia obu skarp są poprawne, co przekłada się na wysokie wartości współczynników stateczności ogólnej (FoS).

Słowa kluczowe: niestateczność, nasyp drogowy, osuwisko, flisz Karpacki, współczynnik stateczności

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1. Introduction

Embankments, as elements of road infrastructure, can be classified as earthworks which are used to build the road surface. Their task is to transfer loads, both dead (the weight of the embankment) and live loads of the vehicles moving along the road sufrace, onto the native soil layers located below.

When designing road embankments, it is necessary to take into consideration the many problems connected with the need for settlement reduction, providing stability and the embankment load-bearing capacity as well as durability. One of the major difficulties is the presence of weak soils under the embankments which may considerably influence the structural stability, especially when it is located on a steep slope. A suitable soil load-bearing capacity under the embankment, characterised by low strength parameters, can be achieved by strengthening it by means of modern geoengineering methods. The incline of the area upon which the embankments are located also significantly influences their stability and durability. This problem causes difficulties in arranging and compacting the material – this requires detailed planning of works; it is also connected with a slower speed of work and an increase of investment costs.

2. Stability and the causes of stability loss in fill slopes and slopes

The stability of the fill slopes, slopes, embankments and excavations is one of the most important problems not only in geotechnical designing but also in evaluating the influence of the potential phenomenon on peoples' safety. As a measure of stability, the general Factor of Safety FoS (coefficient of the equilibrium state) is adopted – this is a ratio of the sum of maintaining forces (friction, cohesion of the material) and the sum of load forces (gravitation and filtration forces). Depending on the value of FoS we can clearly determine whether the analysed slope or embankment are in a state of general equilibrium [11]:

- FoS < 1 – instable slope or embankment
- FoS = 1 – slope or embankment in temporal stability
- FoS > 1 – stable slope or embankment

The surplus of coefficient $F = 1$ determines safety margin.

According to Section V of the Regulations of the Minister of Transport and Marine Economy from 2 March 1999 on technical conditions of public roads and their location (Dz.U.1999 nr 43 poz.430), and concerning load capacity and stability of earth road structures, the Factor of Safety FoS of slopes and embankments should not be smaller than FoS = 1.5.

In general, it may be understood that the slope remains stable when there are no mass movements in it. In reality, stability is a much more complicated problem, influenced by such varied factors as, for example, shape, dimensions and span of slope, water impact, atmospheric conditions or dynamic impact.

Two types of stability and loss of road embankments can be distinguished: loss of stability of the road embankment itself, independent of the influence of soil layers under the embankment; loss of the embankment stability connected with landslides of the soil on which the said embankment is located (slope stability loss).

The most frequent cause of road-embankment instability are the weak soils used to build them and their inappropriate densities. Embankment structures built of various soil layers have a load capacity which when exceeded, causes the whole earth structure to lose its stability.

The other cause of road embankment stability loss is the lack of efficient drainage – this may bring about saturation of the embankment soils with precipitation waters which would then weaken and destroy the embankment.

It also happens that there are badly designed road embankments, with fill slopes that are too steep – this can also cause loss of their stability.

3. Determining general stability coefficients by means of MIDAS GTS NX®

Numerical methods based on an advanced method of solving systems of differential equations, the so-called discretisation into finite elements for which the solution is approximated by specific functions only for nodes of this discretisation. The Finite Elements Method is one of the most frequently used methods based on physical model discretisation. Physical model discretisation means that the continuous medium, which actually is a physical model, is substituted by a discreet model. MIDAS GTS NX® is a numerical programme based on the finite elements method FEM and used to simulate actual phenomena occurring in the soils. Basing on FEM, the programme carries out many kinds of analyses used in geoengineering. It also allows the modelling of complex soil and water conditions.

The method of shear strength reduction is based on the assumption that the movements of the slope are one of the factors leading to shear strength decrease and stress change in the analysed soil medium. Formation of the slide surface, according to this method, develops in a place in which reaching the state of shear stress equilibrium and shear strength is the fastest. In the analysis of stability by the shear strength reduction method, we seek the minimum value of the FoS of the embankment, depending on the shape, loads and boundary conditions. Shear strength and the angle of internal friction diminish gradually until the calculations no longer show convergence. The maximum reduction degree of the above mentioned parameters in the place of convergence loss serves to calculate the minimum FoS. In the calculations, it is assumed that the solid model is an elastic-plastic medium fulfilling Coulomb-Mohr's condition [4].

4. Stability analysis of chosen road embankments

The subject of the authors' studies are two kinds of landslides located in various geotechnical conditions. The aim of the performed calculations was to check stability of the slopes located in Sułoszowa (Krakow district, Małopolska province, Poland) and Lipie (Nowy Sącz district, Małopolska province, Poland). Landslide movements were observed on the slopes recently – this caused damage to provincial roads. These landslides are located in the area of various geological structure. The problem of landslides is also found in other countries, for example, the Czech Republic and Croatia [1, 8].

The first road landslide in Sułoszowa on the map of tectonic units in Poland is located in the Silesian-Cracovian monocline, north west of Krakow. It is mainly made up of rocks such as conglomerates, limestone, dolomites, clays and sandstones. These rocks are slightly inclined towards the north east. Chalk formations developed as marls and marl limestone, sands and conglomerates. They occur locally in the form of klippes covering older sediments. We can also distinguish white limestone from the Upper Jurassic era forming rocks characteristic of the region of Ojców.

Favourable for the formation of landslides in the Silesian-Cracovian monocline are precipitation waters in the form of surface and underground water state changes. Among these conditions, we can include infiltration of precipitation waters as the main cause of weakening strength properties, erosion undercutting and abrasion. A complex tectonic structure of the Silesian-Cracovian monocline area is also influential in landslide formation. Rock layers subsiding at the same angle and in one direction are favourable for the gravitational movements of the earth masses.

Another landslide is in Lipie, in the area of the Flysch Carpathian (the northern rim of the Western and Eastern Carpathians). Polish Flysh Carpathians form a region especially predisposed to landslides formation. Generally, the structure of the region consists of alternative layers of sandstone and shales and to a lesser extent mudstone, conglomerate, clay, marl and limestone. In the structure of the Carpathians are also worth mentioning about complicated tectonics. Numerous faults of different displacement sizes divide the rock mass into two blocks. Strong tectonic violation are connected with the movement of rocks in the fault areas and with clear underthrusts.

Landslides in this area of Poland may also appear due to the area sculpture dynamics occurring as a result of big slopes and river valley slope inclinations. Undoubtedly, susceptibility to landslides in this area is influenced by complex hydrological and meteorological conditions. Violent rainfalls, spring melts or long, humid and cool periods lasting for several months are especially dangerous. Water is then stored in detrital covers and bedrock – this causes deep structural landslide formations.

5. Sułoszowa landslide stability analysis

The area covered by the studies is located in the village of Sułoszowa in the Krakow district of Małopolska province – the area is a part of the Pieskowa Skała settlement.

In this area in September 2014, a fragment of the embankment was damaged. The fragment was located at the junction of a provincial road with the Prądnik stream in Wernyhora street. The slope fragment runs along an asphalt road within which there is the Prądnik stream. The fill slope forms the left side of the stream valley slope and in this area, it is partly crumbled. At the top of the embankment, on its northern side runs, the provincial road (Fig. 1).

The studied area is situated within the Krakow, Częstochowa and Olkusz (Ojców plateau) uplands. In respect of geology, the discussed landslide is located within the Silesian-Cracovian monocline. The region is formed of Jurassic limestone with considerably developed karst phenomena. The surface is slightly wavy and covered with a substantial layer of loess and outliers.

The embankment is located at the bottom part of the slope. Its height is around 3.1m and the whole length is around 20 m. A segment of around 8m was strengthened with a lime retaining wall of about 1.5 m in height at the bottom part the embankment. The inclination of the embankment in the places without the retaining wall varies from 55° to 60°. In the upper areas of the fill slope and in the segment with the retaining wall, the inclination angle is 70–80° [5].

Fig. 1. A damaged fragment of the fill slope and the Prądnik (Białucha) stream undercutting it (Source: [5])

Computer model of the embankment and material data used to model are presented below (Fig. 2, tab. 1).

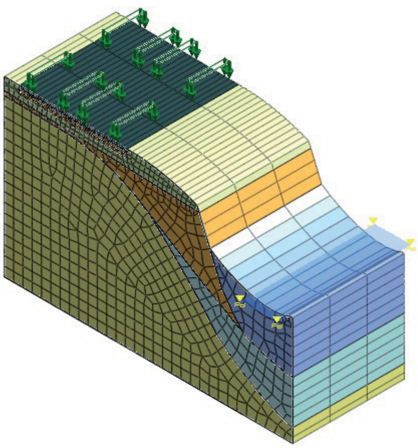

Fig. 2. Computer model of a road embankment (Source: the authors' own study)

Table 1

Geotechnical parameters of the layers in a calculation model

Marking	Soil symbol	Material model	Layer symbol	Poisson's ratio [–]	Natural humidity [%]	Volumetric density [t/m³]	Cohesion [kPa]	Internal friction angle [°]	Young modulus [kPa]
	–	Elastic	**road**	0.2	–	2.50	–	–	50 000
	–	Elastic	**substructure**	0.2	–	2.20	–	–	50 000
	–	Elastic	**gabion**	0.2	–	2.80	–	–	100 000
	–	Elastic	**strengthened substratum**	0.2	–	1.70	–	–	300 000
–	–	Elastic	**geosynthetics**	0.2	–	1.70	–	–	130 000
	nB	Mohr-Coulomb	**I**	0.3	20	1.85	0	10	50 000
	G^{Π} (pl)	Mohr-Coulomb	**IIa**	0.3	13	2.06	19	13	20 000
	G^{Π} (pl)	Mohr-Coulomb	**IIb**	0.3	15	1.94	10	8	20 000
	W	Elastic	**IIIa**	0.2	–	2.20	–	–	2 000 000
	KW	Elastic	**IIIb**	0.2	–	2.20	–	–	2 000 000

(Source: the authors' own study)

Two kinds of slope stability analysis were carried out. The first analysis assumes a dry period (lack of water filtering from waterfall infiltration). The second analysis assumes a wet period (higher ground water level). During these two different periods, the strength and deformational parameters of the soils have not changed. The simulation was performed under drained conditions. Below, the results of stability analysis are presented.

On the basis of the obtained Factor of Safety FoS for the analysed fill slope (FoS = 0.8 – dry period, FoS = 0.7 – wet period) it can be stressed explicitly that it is unstable. In the enclosed figures (Fig. 3, 4, 6, 7) showing the displacements of the analysed solid, it can be seen that the largest deformations appear in the layers of the building embankment (humid silty clays and clays in plastic state). These layers initiate the formation of maximum shear strain zones connected with the appearance of a surface that has the potential to slide (Fig. 5, 8).

Vibrations and quasi-static loads caused by vehicle traffic on the provincial road cause mass movements in the area of the fill slope. Due to its strong inclination and continual washing and cutting by the waters of the Prądnik stream, it is probable that mass movements will develop further. The situation is worsened by the geological structure of the fill slope in the form of coherent soils of low load-bearing capacity and high susceptibility to water influence – this negatively effects the strength of the slope causes increased levels of softness.

Dry period
Factor of Safety FoS = 0.80

Wet period
Factor of Safety FoS = 0.70

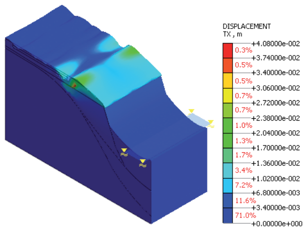

Fig. 3. Horizontal displacement [m] together with deformation (overstated scale) (Source: the authors' own study)

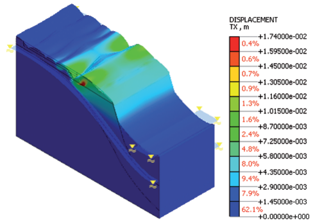

Fig. 6. Horizontal displacement [m] together with deformation (overstated scale) (Source: the authors' own study)

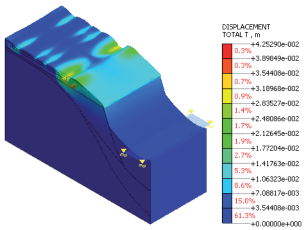

Fig. 4. Total displacement [m] together with deformation (overstated scale) (Source: the authors' own study)

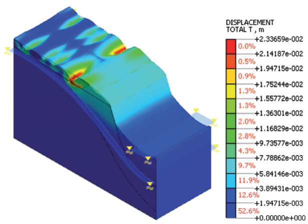

Fig. 7. Total displacement [m] together with deformation (overstated scale) (Source: the authors' own study)

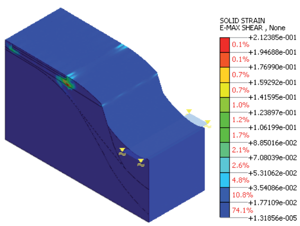

Fig. 5. Maximum shear strains [–] (Source: the authors' own study)

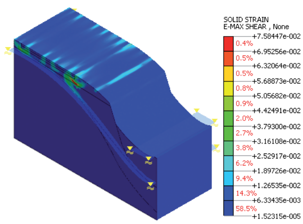

Fig. 8. Maximum shear strains [–] (Source: the authors' own study)

To protect the unstable slope by the Prądnik stream, the priority is to limit washing and undercutting of the slope. An original solution was proposed and verified by calculations in the MIDAS programme. The protection was modelled in the form of a gabion consisting of four baskets of 1.0×1.0 m in dimension. The height of the whole designed structure is 4 m. Additionally, three layers of geosynthetics were laid directly under the building embankment;

their purpose task was to order water conditions in the area being stabilised, and protect it against uneven settling. Each geosynthetic layer was laid on a 30 cm strengthened substratum which could carry vibrations and loads caused by vehicle traffic on the province road and partly increase the load-bearing capacity and durability of the structure. The bottom geosynthetic layer was stretched along the rock layer for a length of 7 m to ensure durable strengthening under the embankment soil. Computer models of the reinforced embankment are presented below (Fig. 9).

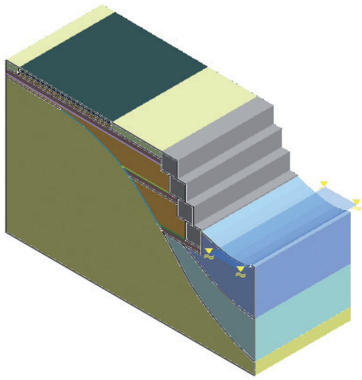

Fig. 9. Computer model of a road embankment
(Source: the authors' own study)

Dry period
Factor of Safety FoS = 4.19

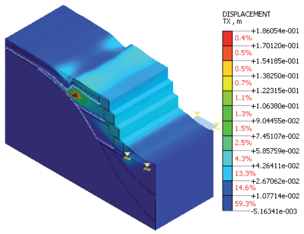

Fig. 10. Horizontal displacement [m]
with deformation (overstated scale)
(Source: the authors' own study)

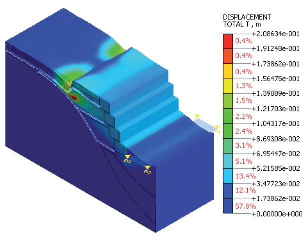

Fig. 11. Total displacement [m]
with deformation (overstated scale)
(Source: the authors' own study)

Wet period
Factor of Safety FoS = 4.09

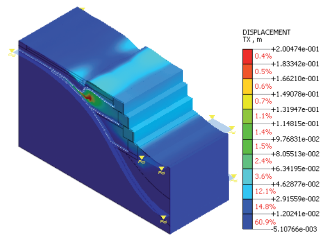

Fig. 13. Horizontal displacement [m]
with deformation (overstated scale)
(Source: the authors' own study)

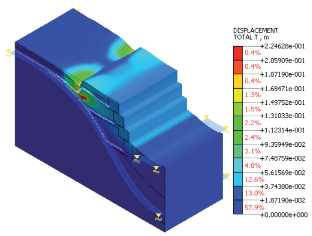

Fig. 14. Total displacement [m]
with deformation (overstated scale)
(Source: the authors' own study)

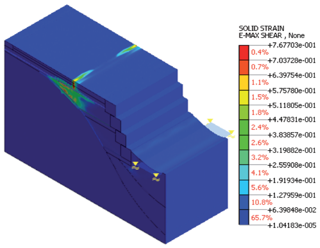

Fig. 12. Maximum shear strains [–]
(Source: the authors' own study)

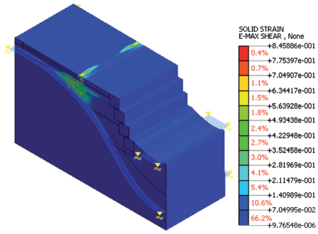

Fig. 15. Maximum shear strains [–]
(Source: the authors' own study)

Having protected the slope with gabions and three layers of geosynthetics on the strengthened substratum layer, it was possible to prevent soil masses from sliding. The solution is definitely favourable as it allowed achieving high Factors of Safety FoS = 4.19 (dry period) and FoS = 4.09 (wet period). The surface susceptible to sliding does not directly endanger the stability of the whole embankment.

As shown in the results of calculations presented in Figures 10–15, all of the used types of protection – gabion, synthetics and the strengthened layer of the soil substratum – fulfil the functions assumed earlier.

6. Stability analysis of the landslide in Lipie

The studied area is located in Lipie, about 3 km south of Gródek nad Dunajcem (Gródek over the Dunajec river), in Nowy Sącz district. The landslide occurred on the south-western shore of Rożnowskie lake, on a provincial road resulting in damage (Fig. 12).

As far as morphology is concerned, the area comprises a slope with a 14° inclination and a western orientation reaching Rożnowskie lake. The slope is cut through by provincial road no. 975 which is a single carriageway with an 8m width and built on an embankment with a height of 4.5 m.

Landslide movement has caused the destruction of a significant proportion of provincial road in recent years (Fig. 16). It is worth mentioning that the discussed landslide occurred within an older and deeper landslide covering a larger area.

The studied area is located within the flysch Carpathians. In the geological structure of the substratum, we can distinguish formations from the Eocene and Quaternary periods. Flysch Carpathians in the region of Poland are most at risk of landslides [7].

The older landslide starts slipped around 13 m over the fill slope above the provincial road. Its dimensions are about 90 m in width and 80 m in length. It is estimated that the landslide in Lipie covers an area of 0.68 ha. The form of the most recent landslide is marked with a main fill slope of up to 2 m in height, located above the access road.

On the basis of archive data, it can be stated that the landslide was already active in the 1950s. In 2001, the landslide caused the breaking of the provincial road. In 1997, 2010 and 2011, its renewal could be observed. In 2010, a geological expert opinion was commissioned to be prepared and it proved that changes within the road traffic lanes were observed for several years and this was connected with road settling. In this period, the road embankment was strengthened with rock fill; however, it did not stop the discussed process [2].

The problem of landslide in Lipie was also dealt with by Stopkowicz & Cała [10].

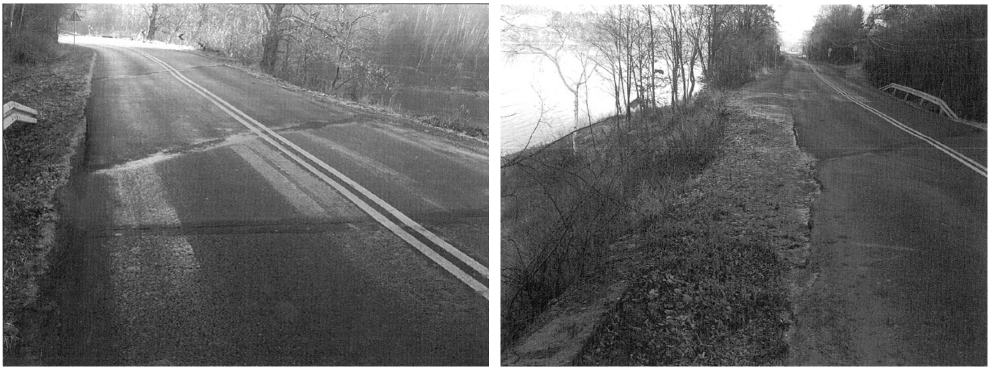

Fig. 16. Damage to provincial road no. 975 as a result of landslide movements (Source: [3])

Three kinds of analysis of slope stability were carried out. The first analysis assumes the occurrence of a dry period (lack of water filtering from waterfall infiltration). The other analysis assumes the occurrence of a transitory period in which there appears, on a moderate level, waters from waterfall infiltrations. The third analysis was carried out under the assumption of the occurrence of a wet period with total saturation of the soil layers. The strength and deformational parameters of the soils remained unchanged. The simulation was performed under drained conditions. The computer model of the embankment and the material data used for modelling are presented below (Fig. 17, Table 2).

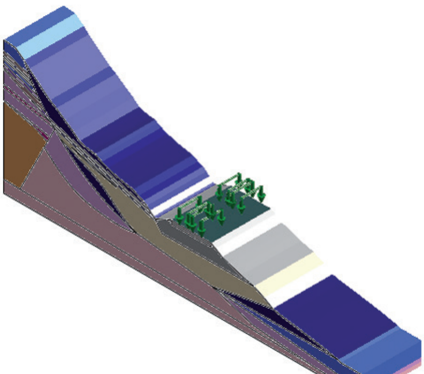

Fig. 17. Landslide computer model (Source: the authors' own study)

Table 2

Geotechnical parameters of the layers in a calculation model

Marking	Soil symbol	Material model	Layer symbol	Poisson's ratio [–]	Natural humidity [%]	Volumetric density [t/m³]	Cohesion [kPa]	Internal friction angle [°]	Young modulus [kPa]
	–	Elastic	**road**	0.2	–	2.50	–	–	50 000
	–	Elastic	**sub-structure**	0.2	–	2.20	–	–	50 000
	–	Elastic	**concrete wall**	0.2	–	2.50	–	–	50 000
	–	Elastic	**strengthened substratum**	0.2	–	1.70	–	–	300 000
–	–	Elastic	**geo-synthetic**	0.2	–	1.70	–	–	130 000
	nB	Mohr-Coulomb	–	0.3	20	1.85	0	10	50 000
	G, Gz,	Mohr-Coulomb	**Ia**	0.3	21	1.85	7	9	30 000
	G, Gz,	Mohr-Coulomb	**Ib**	0.3	16	2	8	15	50 000
	G, Gz	Mohr-Coulomb	**IIa**	0.3	13	2.15	18	15	22 000
	Pg, πp	Mohr-Coulomb	**IIb**	0.3	10	2.20	26	17	30 000
	Pd, Pπ	Mohr-Coulomb	**III**	0.3	6	1.85	0	29	38 000
	KW	Mohr-Coulomb	**IV**	0.3	12	2.00	57	12	20 000
	KW	Mohr-Coulomb	**V**	0.3	5	1.80	0	33	114 000
	SM	Mohr-Coulomb	**VI**	0.2	–	2.48	250	0	26 000
	ST	Mohr-Coulomb	**VII**	0.2	–	2.52	250	0	100 000

(Source: the authors' own study)

On the basis of the performed analysis and resulting stability coefficients (FoS = 0.73 – dry period, FoS = 0.74 – transitory period, FoS = 0.50 – wet period) it can be stated that the discussed embankment is unstable. The figures below (Fig. 19, 22, 23, 25, 26) show displacements of the analysed embankment. The highest displacement values can be seen in the layers of the building embankment, clayey sands and clays. In the figures showing maximum shear strains (Fig. 21, 24, 27) the sliding surface was clearly marked – this shows that soil shear may take place in the layers mentioned earlier.

As the analysed road embankment is located in an area prone to landslides, it is constantly endangered by landslide activity. In periods of heavy rainfall, due to dramatic increases of water flow pressure within the colluvium, it may happen that the values of the soil substratum strength parameters reduce resulting in landslide movements. Undoubtedly, the presence of deposits sensitive to water activity and which quickly become waterlogged in the substratum is favourable for this process. In the autumn and spring periods, when intensive rainfalls and melts occur, the landslide movements can increase.

As we are dealing with an unstable road embankment located within an older and deeper landslide, a protection strategy was designed in the form of a reinforced concrete retaining wall (slab and angle) located on the Rożnowskie lake side (Fig. 18). The designed retaining wall, sunken to a minimum of 1.5 m below the lowest slide surface, has to transfer the pressure on the soil, caused by vehicles on the provincial road. Additionally, three layers of geosynthetics were laid directly under the building embankment – this was to order water conditions and limit uneven settlings. Each of the three layers of geosynthetics was laid on a 30 cm thick strengthened substratum layer, the purpose of which was to help to transfer vibrations and quasi-static loads caused by vehicle movement on the provincial road. For construction reasons, the layer of geosynthetics was also laid along the whole length of the retaining wall together with the bottom slab. A water drain was located above the bottom slab.

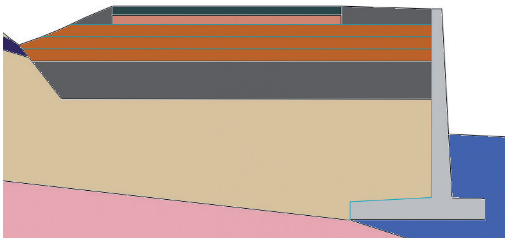

Fig. 18. Approximation of road embankment strengthening (2D view)
(Source: the authors' own study)

The dimensions of the retaining wall:

- height 8.9 m
- wall thickness 45–85 cm
- width of base – 5.85 m
- thickness of bottom plate – 90 cm

Proposed solution made the fill slope adequately protected. Stability coefficients increased significantly for all three calculation cases – FoS = 2.36 (dry period), FoS = 2.05 (transitory period), FoS = 1.66 (wet period) – this explicitly proves the accuracy of the realised solution. The displacements of the analysed solid (Fig. 28, 29, 31, 32, 34, 35) and maximum shear strains (Fig. 30, 33, 36) show that if the loss of stability occurs, then it will occur behind the road embankment strengthened by the retaining construction.

7. Summary

At present, there are many tools on the market, which allow the engineers to predict the possible behaviour of the soil masses. The dynamic development of software programs used for the analyses of embankment and slope stability not only allows the checking road embankments with regard to their predisposition to landslide activity but also to carry out analyses of the designed protection solutions.

The presented examples show that the best solution is to use a system of protection combining several methods. The analysed embankments were located in totally different geotechnical conditions. The first analysed example is in the Silesian – Cracovian monocline, the second is in the Carpathians. The road embankment in the flysch Carpathians, in Lipie, revealed higher levels of instability. This proves the view that there are serious landslide risks connected with the geological structure in the Carpathian flysch. Such a geological structure causes risks to road infrastructure. The proposed and modelled protections of road embankments presented in the works in Sułoszowa and Lipie combine the methods of protective construction with geosynthetic strategies and allow the obtaining of satisfactory stability indices and at the same time, to order water conditions. The final effect is not only to ensure the stability of the embankment but also the functionality of the structure located on it.

The problem the authors tried to present is rather a universal one. Correct and carefully built road embankments that use an adequate range of materials and ensuring proper drainage may, in given location conditions, prevent landslide movements. However, there may be cases when the road constructions are in advance endangered by stability loss. The knowledge and experience of the designer and people involved in the embankment and slope protection process then become especially important.

Dry period
Factor of Safety FoS = 0.73

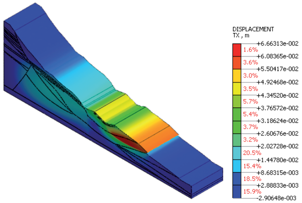

Fig. 19. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

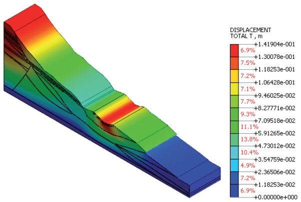

Fig. 20. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

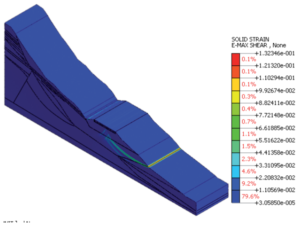

Fig. 21. Maximum shear strains [–] (Source: the authors' own study)

Transitory period
Factor of Safety FoS = 0.74

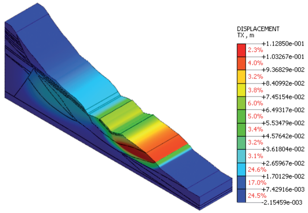

Fig. 22. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

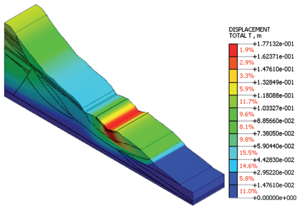

Fig. 23. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

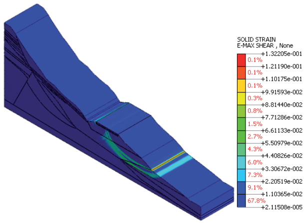

Fig. 24. Maximum shear strains [–] (Source: the authors' own study)

Wet period
Factor of Safety FoS = 0.50

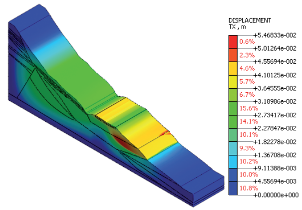

Fig. 25. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

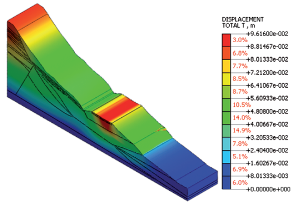

Fig. 26. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

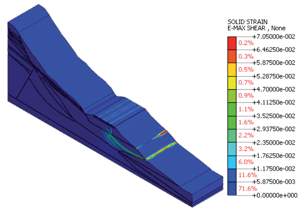

Fig. 27. Maximum shear strains [–] (Source: the authors' own study)

Dry period
Factor of Safety FoS = 2.36

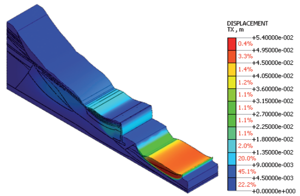

Fig. 28. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

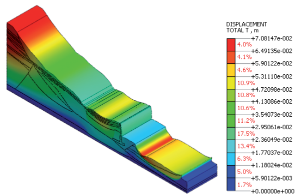

Fig. 29. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

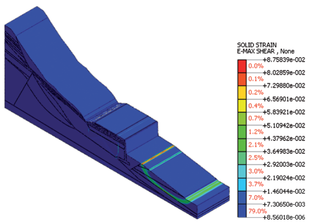

Fig. 30. Maximum shear strains [–] (Source: the authors' own study)

Transitory period
Factor of Safety FoS = 2.05

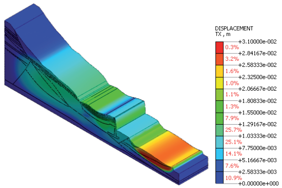

Fig. 31. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

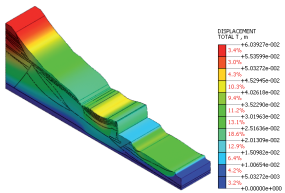

Fig. 32. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

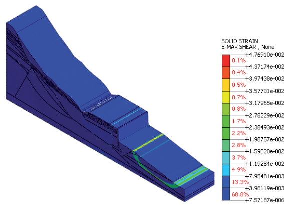

Fig. 33. Maximum shear strains [–] (Source: the authors' own study)

Wet period
Factor of Safety FoS = 1.66

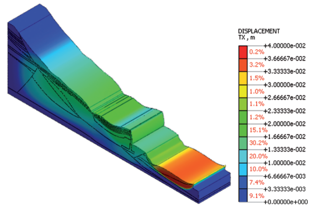

Fig. 34. Horizontal displacement [m] with deformation (overstated scale) (Source: the authors' own study)

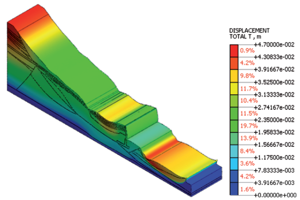

Fig. 35. Total displacement [m] with deformation (overstated scale) (Source: the authors' own study)

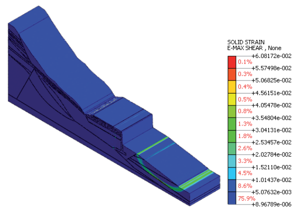

Fig. 36. Maximum shear strains [–] (Source: the authors' own study)

References

[1] Baron I., Agliardi F., Ambrosi C., Crosta G. B., *Numerical analysis of deep-seated mass movements in the Magura Nappe; Flysch Belt of the Western Carpathians (Czech Republic)*, Natural Hazards and Earth System Science, 5(3), 2005, 367–374.

[2] Białek M., *Analiza stateczności wraz z projektem zabezpieczenia nasypów drogowych zlokalizowanych w różnych warunkach geotechnicznych*, M.Sc. Thesis, Kraków 2015.

[3] *Dokumentacja geologiczno-inżynierska dla potrzeb zabezpieczenia osuwiska w ciągu drogi wojewódzkiej nr 975 w miejscowości Lipie, gm. Gródek nad Dunajcem*, Zakład usług Geologicznych i Projektowych Budownictwa i Ochrony Środowiska – GEOTECH, Kraków 2011.

[4] *MIDAS GTS NX, 2014*, New experience of Geo-Technical analysis System, Manual documentation, 2014.

[5] *Opinia geotechniczna celem zabezpieczenia skarpy na styku drogi wojewódzkiej nr 773 w Sułoszowej*, Przedsiębiorstwo Geologiczne S.A., Kraków 2014.

[6] *Problematyka osuwisk w budownictwie komunikacyjnym*, Zeszyty Naukowo-Techniczne Stowarzyszenia Inżynierów i Techników Komunikacji Rzeczpospolitej Polskiej Oddział w Krakowie, Materiały Konferencyjne, No. 8 (Issue 144), PiT, Kraków 2009.

[7] Raczkowski W., *Landslide hazard in the polish Flysch Carpathians*, Studia Geomorphologica Carpatho-Balcanica, Vol. XLI, 2007, 61–75.

[8] Soldo B., Ivandić K. & Orešković M., *Numerous landslides and landslide restorations – an example*, Acta Montanistica Slovaca Ročník 17 (2012), číslo 4, 257–262.

[9] Srokosz P. E., Damicz J., Bartoszewicz A., *Utrata stateczności skarpy nasypu drogowego*, XXIII Konferencja Naukowo-Techniczna Awarie Budowlane, Szczecin–Międzyzdroje 2007.

[10] Stopkowicz A., Cała M., *Analiza stateczności zboczy zlokalizowanych we fliszu karpackim z zastosowaniem metod numerycznych*, XXVII Zimowa Szkoła Mechaniki Górotworu, Zakopane 2004.

[11] Ukleja J., *Geotechniczne aspekty stabilizacji obszarów zagrożonych osuwiskami*, Oficyna Wydawnicza, Opole 2013.

[12] Wysokiński L., *Metody prognozowania i zabezpieczania osuwisk*, XXV Konferencja Naukowo-Techniczna Awarie Budowlane, Międzyzdroje 2011.

[13] Wysokiński L., *Ocena stateczności skarp i zboczy. Zasady wyboru zabezpieczeń. Instrukcje, Wytyczne, Poradniki 424/2011*, Instytut Techniki Budowlanej, Warszawa 2011.