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Protection of a building against landslide. A case study and FEM simulations

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Abstract. The main objective of this work was to present a successful stabilization action of a building structure in an active landslide. Firstly, history of the case and a FEM simulation explaining ensuing situation are presented. Then different structural measures to stabilize the whole system are discussed. The structural solution of the problem (pile system reaching solid rocky zone) is presented in more detailed way. The estimation of forces acting on the structure, caused by an unstable soil mass, being crucial for the design of stabilizing structure is described.

Key words: landslide, numerical analysis, structural measures of landslide protection.

1. Introduction

According to Cornforth [1], landslides are complex phenomena. The main factors which have a significant influence on their behavior are changes in the geometry of the slope, changes in the water conditions and increase of the external loads (Wysokiński [9], Sarah and Daryono [5]). Due to the complexity of the problem, classic engineering methods (based on limit equilibrium concept) for stability assessments often fail. In this situation, numerical analysis of the stability, (usually performed with the use of the c–ϕ reduction method) can be useful (Truty et al., [6], Wysokiński [9], Zheng et al. [10], Ozbay and Cabalar [4]), giving possibility to model landslide problems very close to the reality.

2. History of the case

The study area is located near Cracow in southern Poland, in the marginal zone of the Carpathian Mountains. Bedrock (paleogene age) is composed of clastic rocks. These are mainly slates, mudstones and sandstones. Limestone and marl are occasionally found. On these rocks, which are located at a depth of about 18–30 m, residual deposits can be found (different varieties of clay, silty and sandy clays). Within subsurface soils there is groundwater filtrating, which can cause a change in ground conditions. Such layers can be potential slip surfaces.

The construction of a building on a slope (about 140 m high, average inclination about 1:10) was planned. The terrain (according to Polish Geological Institute website http://geoportal.pgi.gov.pl/SOPO) was classified as potentially landslide endangered (no. 1 in Fig. 1).

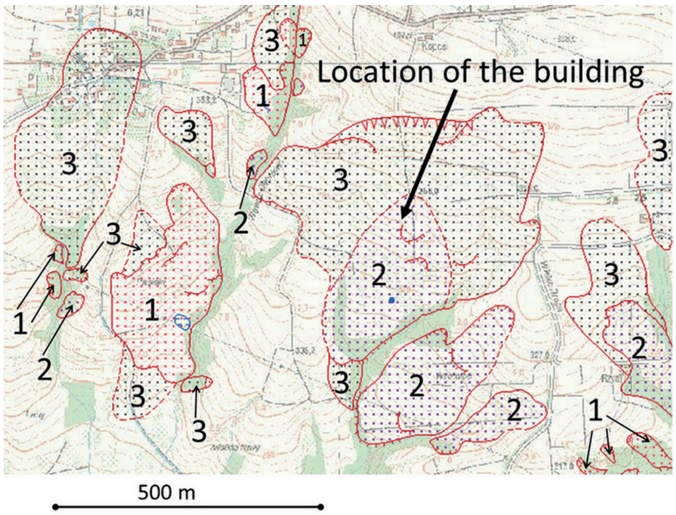

Fig. 1. Location and topography of the study area (after http://geoportal.pgi.gov.pl/SOPO) with constantly active (1), periodically active (2) and non-active (3) landslides marked

Active landslide zones are marked on the map (as no. 2), but it is worth to note that the landslide which is described here was added after the described events. As first action, performed in order to level the terrain in the site, a layer of clayey soils with varying depth up to about 6 m was placed on the area (forming an artificial slope with inclination of about 1:2). Then, in the autumn of 2008, after heavy rainfalls, soil mass movement was observed for the first time. It was initially (and not quite correctly) identified as only the stability loss of the newly created artificial slope. Thus geotechnical evidence (performed at May, 2009) gave information only about soil conditions on shallow layers of the soil, to the depth of about 4 m under initial (prior to earthworks) ground surface. Those layers were formed from silts and clays. The second hypothesis (later proved by inclinometric monitoring and numerical simulations, described

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below) was that a deep, rotational landslide has been activated by overloading of the existing natural slope by the creation of the artificial slope. An additional reason of the landslide activation was an inflow of the deep groundwater (due to heavy rainfall and snow melting).

In 2010 the design and construction of the high-quality residential building (2-storey + cellar, horizontal dimensions about 14×33 m) was performed.

The structure consisted of a base plate (raft), floor plate and cellar walls (heavily reinforced concrete, 30 cm thick) which are forming stiff, monolithic box. Moreover, it was founded on the set of 16 barrettes ($4.5 \div 8$ m length, measured from the bottom of base plate, 3.0 m width and thickness 0.5 m), introduced by designers with the intension to prevent uneven settlements of the soil layers used to level the terrain. Additionally, in the distance of several meters from the building a retaining wall, also founded on the barrettes, was built. In 2010 and 2011 (especially after heavy rainfalls) activity of the landslide increased. A fault line on the ground surface, (Fig. 2) was observed. From July 2010 geodetic monitoring of the building began. 21 benchmarks and 3 inclinometers were installed. In August 2010 new geotechnical documentation was ordered, giving information about soil to the depth of about 35 m. Subsoil consisted of silts and clays, firm rock sub-grade (limestone) was found at about 30 m under the terrain level. Groundwater surface varied in different points from 2 to 7 m under the initial terrain level. Soil mass above groundwater surface is heavily saturated, with saturation ratio S reaching the value of 0.6.

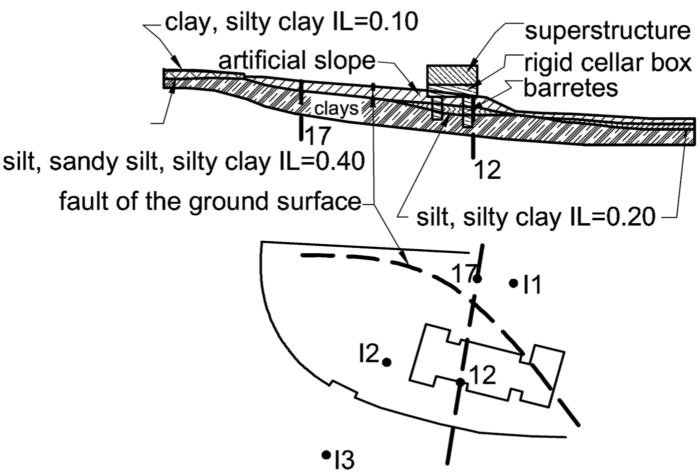

Fig. 2. An overview of the situation and representative geotechnical cross-section

3. Analysis of the monitoring results

Geodetic monitoring (inclinometry and surface geodesy) shows that building and its surrounding constantly move downward the slope, with speed of about 1 cm per month. Cumulative displacements from July to December 2011 reaches about 145 mm in the vertical direction (settlement) and about 250 mm in the horizontal direction (Fig. 3b). Such large displacements (especially horizontal) clearly shows that subsoil is working in the ultimate state, stability loss occur. As it can be seen the sliding surface is located about 17 m under the terrain level (333 m.a.s.l) in the vicinity of the inclinometer I-2 and about 20 m (324 m.a.s.l) in the vicinity of the inclinometer I-3 (Fig. 3a). Sliding surface does not reach the rocky sub-grade. No sign of stability loss are observed in inclinometer I-1. Observed fault of the terrain (between inclinometers I-1 and I-2, Fig. 2) gives the ability to locate the intersection of the upper part of sliding surface with terrain. Observed uplift of the terrain below the building allows to locate the intersection of the lower part of sliding surface with terrain. According to the inclinometric measurement landslide moves in a diagonal

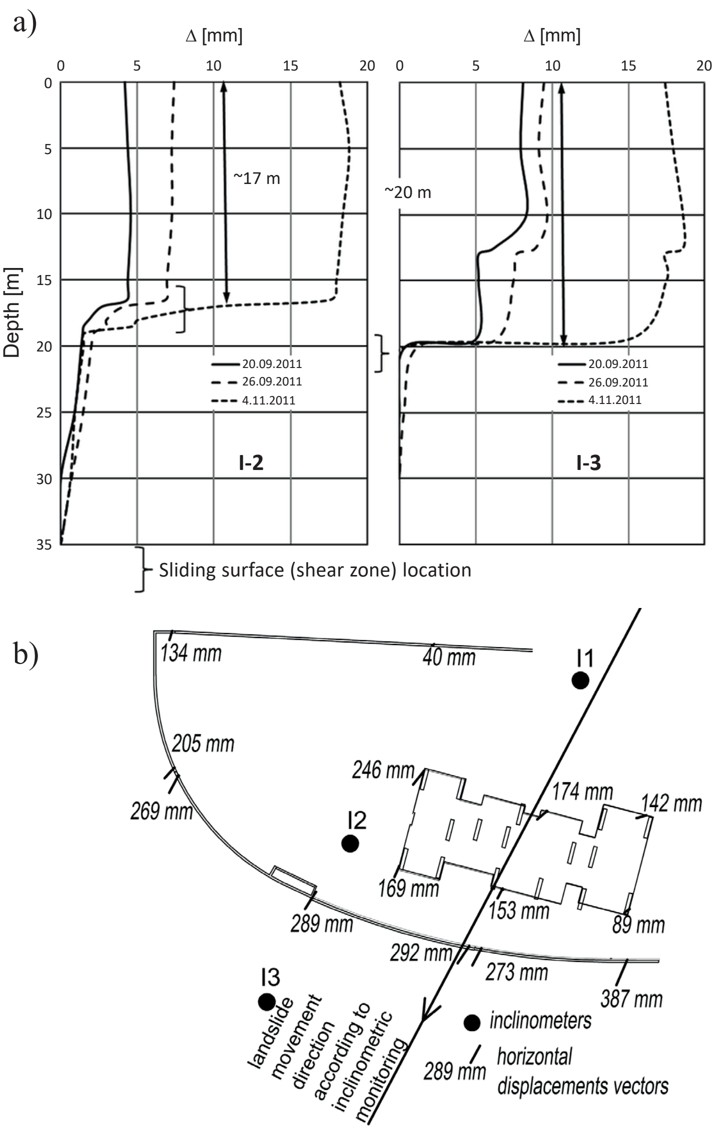

Fig. 3. Inclinometric results (a) and surface geodesy (b) at the end of 2011

direction with respect to transverse axis of the building (angle between landslide direction and transverse axis of the building is about 15°).

4. Numerical simulations of a landslide – back analysis

Since winter 2012 involvement of the authors as experts/designers of protection works started. As a first step towards better understanding of the observed phenomena (stability loss of the slope) numerical simulations by finite element and c–ϕ reduction method, using ZSoil.PC® code, were performed. Analysis was performed assuming 2D plane strain model, which was chosen as an approximation of 3D reality, mainly because its efficiency. The soil continuum was treated as an elastic-plastic one, with Mohr-Coulomb yield condition.

Mohr-Coulomb model was chosen because of its simplicity and usability in an ultimate state problems. The usage of more sophisticated soil model (hardening small strain model for example) would be also possible, but it requires some additional soil tests, mainly related to proper identification of the nonlinear stiffness. Such tests were not available and the Authors were forced to act in the situation of limited information about soil.

Influence of an underground water pressure field in saturated or partially saturated zone was taken into account, with enhancements of the flow theory given in Van Genuchten [8], basing on observed water table.

The applied c–ϕ reduction algorithm is an objective method of a slope stability analysis, because it does not require any assumption of a failure mechanism (location and shape of the sliding surface). Sliding surface, understood as the zone of where localized shear deformation appears, comes out as the result of the algorithm, which consists of a series of boundary problem of elastic-plastic analysis, govern only by gradually descending values of strength parameters (with applied loads kept as constant):

$$
\begin{aligned}
&c^{(i)} = c / SF^{(i)} \\
&\tan\varphi^{(i)} = \tan\varphi / SF^{(i)} \\
&SF^{(i)} > 1 \\
&SF^{(i)} = SF^{(i-1)} + \Delta SF,
\end{aligned}
\tag{1}
$$

until divergence of Newton type iteration happens. This event concerning numerical model corresponds to (or is the image of) physical phenomena of loosing equilibrium by the soil massive. In standard version of the method, i.e. when c, ϕ are treated as known, the final value of SF is safety factor estimation and deformation accompanying loss of equilibrium is treated as a sought, most probable, failure mechanism. More detailed description of the c–ϕ reduction method may be found in ZSoil documentation (Zimmermann et al. [11]) or in works by Griffiths and Lane [2] and Matsui and San [3].

More examples of usage of this method in landslide stability analyses are given by Truty et al. [6], Zheng et al. [10], Ozbay and Cabalar [4], Berti et al.[12].

Here, logic of c–ϕ reduction method is somehow reversed, as some information concerning geometry of the failure mechanism is available from inclinometry and surface geodesy, but values of soil strength parameters (cohesion c and internal friction angle ϕ) of the deeply placed weak layers were uncertain (as its estimation was based on liquidity index IL, only). Thus, back analysis was used to find such c and ϕ for the deep subsoil (treated as homogenous), which result in stability loss mode (sliding surface localization) close to observed in reality, with stability factor $SF \cong 1.0$. For shallow layers of the subsoil geotechnical parameters were taken from existing geotechnical evidence. Material zones (geomorphology taken from VIII–VIII geological cross section, see Fig. 2) in subsoil and built-in structures in the FE-model prior to protective action are shown in Fig. 4. Performed computations shows that for parameters of the deep subsoil $c = 10$ kPa and $\phi = 8°$ obtained $SF = 1.09$ and stability loss form (sliding surface) is very close to observed.

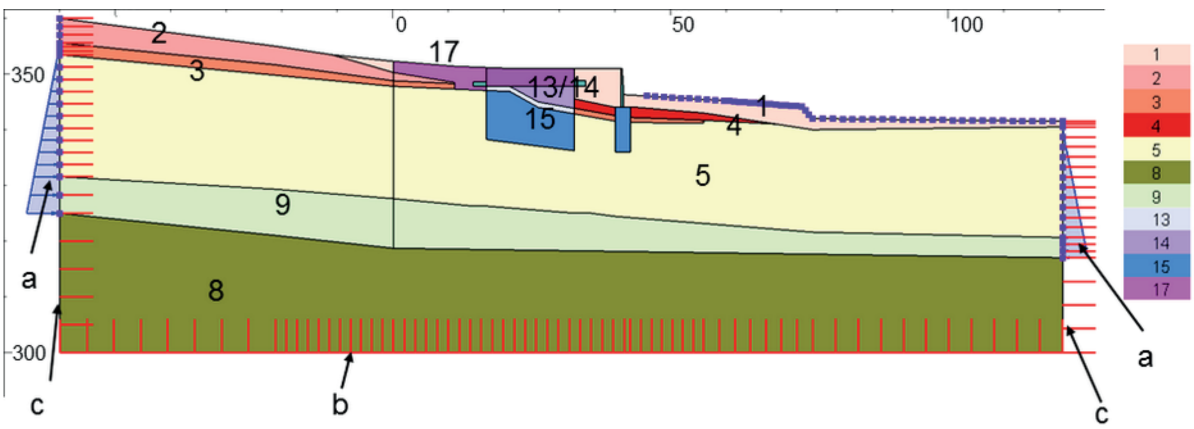

Fig. 4. Material zones distribution and boundary conditions for solid and fluid phase in the cross-section. Zone a – pressure b.c. (Dirichlet type), zone b – no flow condition (von Neumann type), zone c – switchable b.c. (for the suction: $p > 0 \Rightarrow q_n = 0$, for $p \le 0$ free flow), for solid phase – standard geotechnical kinematic b.c for subsoil, static b.c. for upper surface

Such result leads to a slight correction of parameters of the deep subsoil (to the critical values c_{crit} and ϕ_{crit}):

$$c_{crit} = 10/1.09 = 9.17\ \text{kPa},$$
$$\tan\varphi_{crit} = \tan\varphi/SF = 8°/1.09 = 0.14/1.09 = 0.128, \quad (2)$$
$$\varphi_{crit} = 7.31°.$$

Table 1
Soil parameters used in the analysis

Zone ID	Material	E [MPa]	γ [kN/m^3]	c [kPa]	ϕ [°]
1 (IL)	Artificial slope	25.0	19	15	12
2 (IL)	Clay, silty clay, IL = 0.10	36.5	21	35.5	20
3 (IL)	Silt, sandy silt, silty clay, IL = 0.40	13.4	20.5	10.7	11.6
4 (IL)	Silt, silty clay, IL = 0.20	20.5	21	17	14.8
5 (IL)	Deep subsoil (initial values), IL = 0.55	15	19	10	8
8	Rock (limestone)	500	25		
13 (*)	Silt, sandy silt, silty clay, IL = 0.40 + barrettes	1213	20.5	261	11.6
14 (*)	Silt, silty clay, IL = 0.20 + barrettes	1219	21	267	14.8
15 (*)	Deep subsoil + barrettes	1214	19	260	8
9 (IL)	Deep subsoil, clay, IL = 0.15	32	22	28	18
17	Artificial slope – weak layer	25	19	5	4

(IL) – parameters c, ϕ estimated from measured liquidity index, using the method recommended by Polish standard PN-81/B-03200.
(*) – stiffness and strength parameters (X = E,C) estimated like for composite (barrettes + soil) according to approximated formula X = Vb*Xb + Vg*Xg (volumetric homogenization).

The simulated mechanism of stability loss (before protection), i.e. deformation localization zone, is visualized as color contour maps of displacement norm $\|\mathbf{u}\|$ and the deformed mesh. Both are shown in the Fig. 5. The transparent FE-mesh marks stable zones while blue, green and yellow means possible shear zone (failure surface), the red color represents a rigid-movements zone.

It is compared to location of sliding surface according to inclinometers. Obtained from the numerical back-analysis parameters of the deep subsoil were close to reality and can be used in design process of the stabilization of the building.

In the presented methodology of the back-analysis of the landslide initial values of cohesion c and internal friction angle ϕ of the deep do not influence on final values of c_{crit} and ϕ_{crit}, used in later analysis of the structural protection of the building. Similar approach is used by Berti et al. [12].

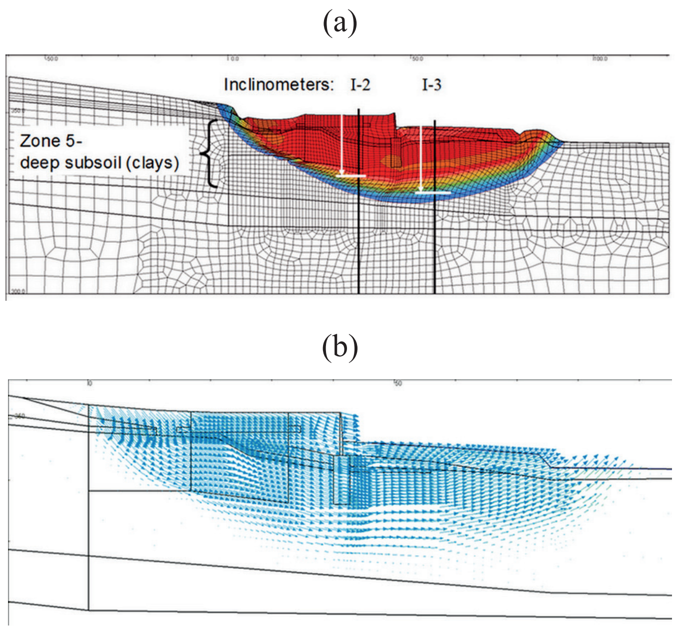

Fig. 5. Simulated and observed shape of sliding surface. For the state of loosing equilibrium a) map of a norm of displacements b) displacement vectors

The c–ϕ reduction method easily handles under-ground water pressure field (two phase media formulation) and the presence of built-in structural elements. Both options turn out to be essential in the presented case.

In order to find how groundwater level influences stability of the slope a parametric study was performed. In it, the groundwater level is changed from ΔH = –5.0 m below the one recorded in the geotechnical evidence to ΔH = 1.5 m above it and assumed as pressure boundary condition in FE model shown in Fig. 4. For every 0.5 m of the water level variation a stability analysis was performed, taking material strength parameters of the deep subsoil for which $SF = 1.0$ ($c_{crit} = 9.17$ kPa, $\phi_{crit} = 7.31°$). The obtained results (Fig. 6) showed that (for this particular case) influence of the groundwater level is noticeable, but not considerable. For ΔH = –5.0 m stability factor increases

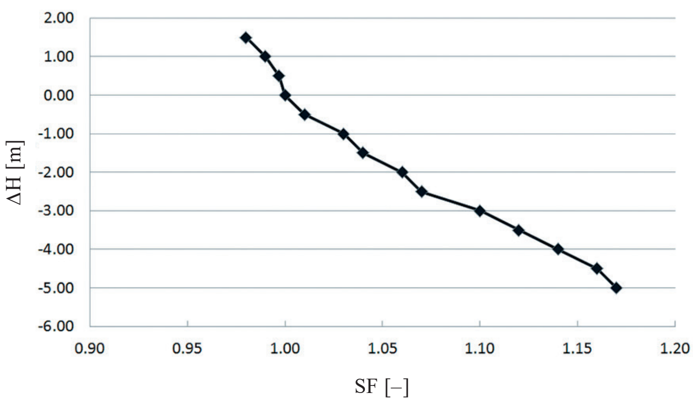

Fig. 6. Influence of the groundwater level change ΔH on the stability factor of the slope

only to $SF = 1.17$. The conclusion is, that dewatering of top soil layers in the considered slope (by drainage around the whole area), would not guarantee stability with the reasonable margin of safety factor. On the other hand dewatering of these deep soil layers, where the sliding surface and mass movement were recorded, seemed to be hardly feasible, which turn as towards undertaking some structural action.

5. Design of the building stabilization structure

The situation, when the nature of the problem has been recognized, was as follows: – structure of the building constantly moves, thus landslide is active, but itself (so far) does not exhibits any signs of internal damage. Nevertheless, some action should be undertaken to stop this process, as it does not allow the proper functioning. Firm decision of the investor was not to withdraw from the project, but to finish construction of the building with an appropriate protection. Because of the size of the landslide, its depth, and the limited area of property (where any action could be undertaken), protection of the whole slope was assessed as not viable. Taking this into account, the option of creation of a stiff and sufficiently strong structure, linking building with solid bedrock (which was detected on the depth reaching 30 m), capable to carry forces induced by potentially moving soil mass, was chosen.

Additionally, the principle of a non-violation of integrity of existing building structure was adopted, understood as not to undertake any action leading to changes or damages in already built parts, but its appropriate completing.

Different structural variants with use of pre-tensioned anchors, foundation beams and piles were analyzed, starting from the simplest and lightest one, schematically presented in the Fig. 7. Each was submitted to simplified computational analysis assuming 2D FEM model, during which all static stiffness characteristics of structural elements were recalculated on the slice of 1 m depth. It allowed to estimate roughly basic features of each schema such as: safety factor SF (by c–ϕ reduction method), forces in anchors and other structural elements, displacement of the building.

Schema (a) and (b) – supporting frame along the front of building, single and double row of anchors were rejected because they do not gives high enough SF ($SF = {\sim}1.23 < SF_{\mathrm{MIN}} = 1.3$). Very long anchors (about 60 m) would be required which would

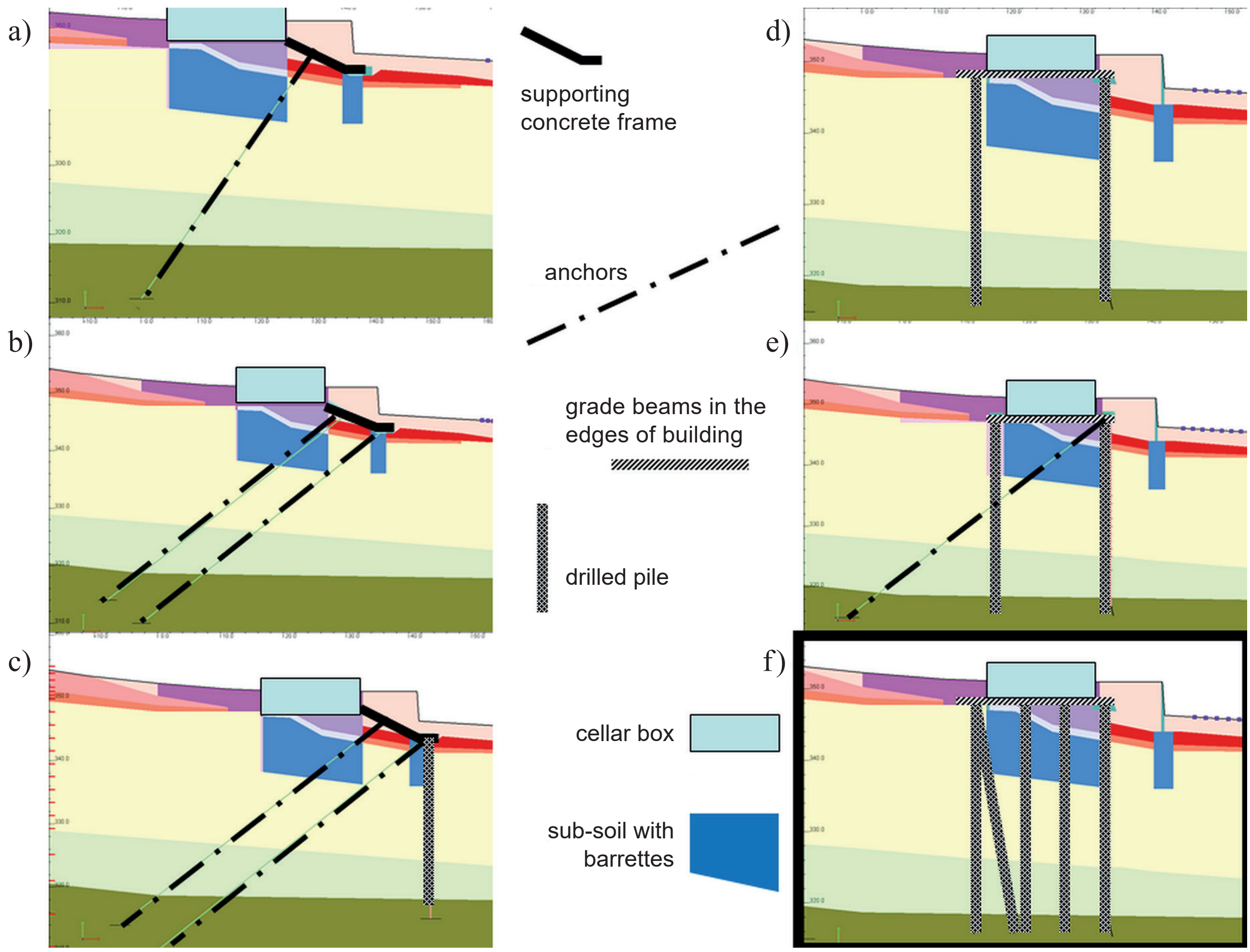

Fig. 7. Schema of initially considered variants of protection: a–e) rejected f) accepted

lead to their low stiffness and finally result in still unacceptable displacements of the building. For schema (c) (the same as (a) and (b) but with additional row of drilled piles stabilizing the structure), similar, not satisfactory, results were achieved. In variants (d), (e) and (f) the main structure are frames placed along lateral edges of the building. Each frame consists of a massive reinforced concrete grade beam and drilled piles of large diameters (1.5–2.0 m) reaching solid rock. Cases (d) and (e) were rejected, because they resulted in a large horizontal displacements of the building (about 0.60 m with $SF = 1.15$), very large bending moments in the frame and low SF between 1.15 and 1.32. Finally, case (f) was accepted for realization. It consisted of two massive frames (A – supported by 9 piles, B – by 3) linked by tension rods. Obtained values of internal forces were reasonable (design of the reinforcement of the concrete elements was possible) and $SF = 1.70$ was enough high to judge that structure would be stable.

Taking into consideration localization and technical obstacles, such as limited possibility of movements of the drilling machine for large diameter piles in very close distance to the building, structural system shown in Fig. 8a was designed.

It was assumed that stiff cellar box is supported by the 9-pile frame, which carried about 50% of vertical load from the building, as in the zone of the landslide a gap might appear between base plate of the cellar box and the subsoil which settles. This assumption was later (i.e. during an inspection after construction) confirmed by the observation. The main difficulty in the design of stabilizing structure was estimating the forces acting on it due to the active landslide. Note, that barrettes placed in the foundation by the original design constitute additional source of forces – they acts as "sails" circumnavigated by the unstable soil mass. Vertical forces from friction between the barrettes and the settling ground also have to be taken into account.

This phenomena was roughly analyzed in subsidiary FEM (2D, plane strain) analyses of the elastic-plastic media surrounding a stiff cross-section of a barrette and a pile, in a manner similar to these, described in Urbański [7], Taheri et al. [13] or Georgiadis et al. [14]. In [7], a pile acting on an elastic-plastic continuum was considered. An acceptable agreement of basic static results (i.e. forces acting on the pile and bending moment) between applied here 2D/3D method and referential, fully 3D, model of pile-soil interaction was shown there. Note, that here the pile position is fixed but its surroundings move, while in [7] reverse situation was analyzed. In both cases relative displacements between the pile and distant points of the surroundings are the source of loads.

Mohr-Coulomb plasticity model parameters of the media were taken as for the deep sub-soil layer (pos. 5 in Table 1), with the slight correction of strength parameters resulting from $c-\phi$ reduction, leading to critical values c_{crit} and ϕ_{crit}. Reaction force (acting on 1m slice) induced by increasing imposed relative displacements are shown in the Fig. 9. Its value for the developed plastic flow ($R_Y = 215$ kN/m) was assumed as an estimation of a maximum load acting horizontally on each pile in the zone above the slip surface. Finally, practical realization of the load from moving soil mass in 3D frame model was done by the following methodology: a set of horizontal connectors (in a distance $a = 1$ m) with elastic-plastic characteristics, i.e. ultimate load $f_Y = R_Y$ and stiffness $k = EA/l = 10\,000$ kN/m (obtained as bilinear approximation of relationships presented in Fig. 9) was introduced to the beam element representing the pile. Displacements ΔU in the direction of the landslide were imposed on external nodes of these connectors which were

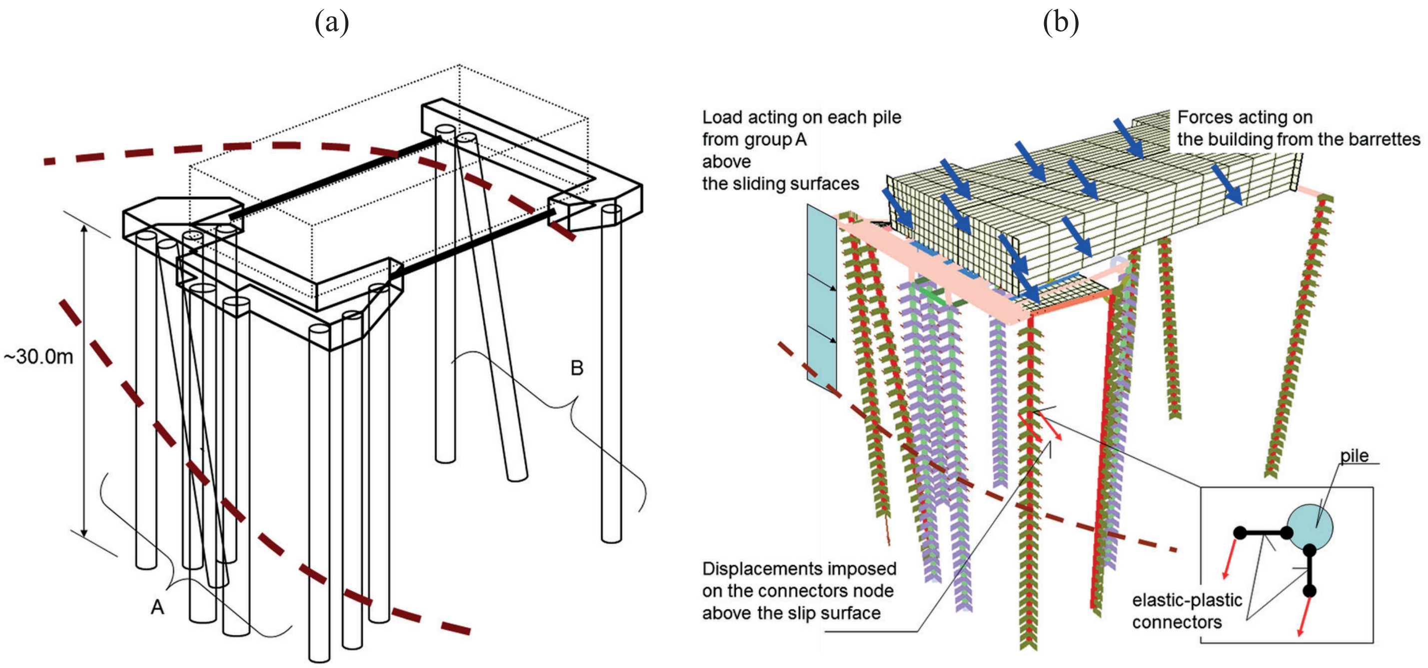

Fig. 8. a) An overview of the supporting structure; b) Static schema of 3D frame system and its load

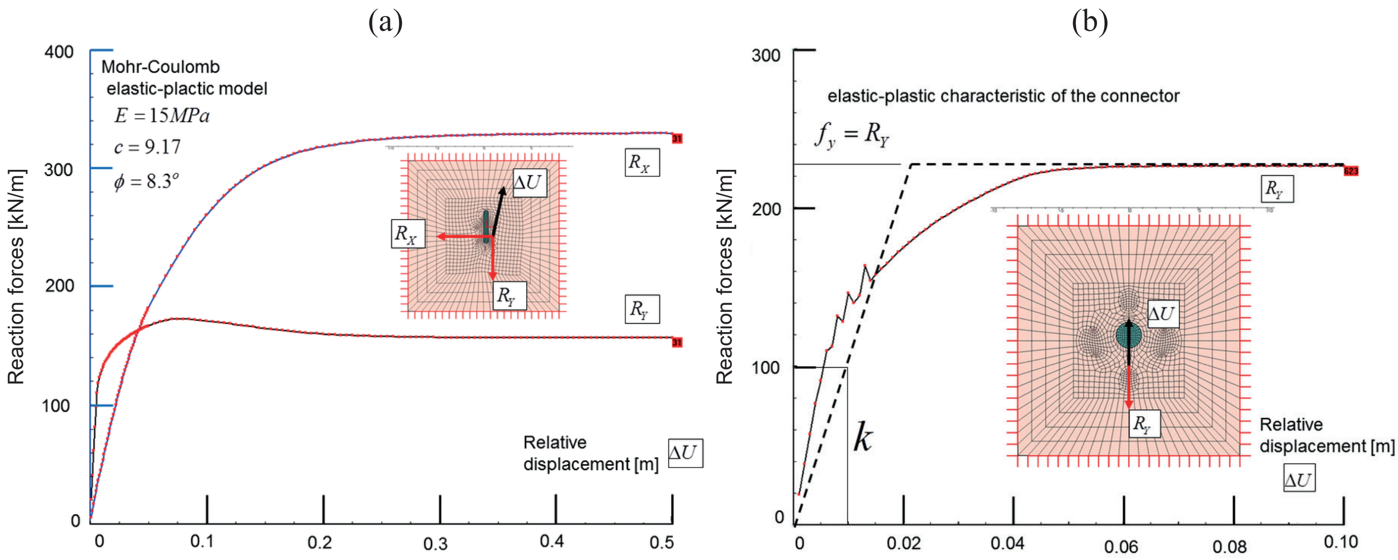

Fig. 9. Forces acting on 1 m length of a) the barrette b) D = 1.8 m pile, with setting the elastic-plastic characteristics of the connectors

placed in moving soil mass, above the slip surface, while these below it were fixed (displacement-driven approach). Note, that assuming this manner of computing, stiffness of the whole frame system has some influence on the load distribution along the pile.

In the Fig. 10 results of the analysis of the whole frame system are shown. The analysis did not concern the cellar box, so that the plate finite elements visible on the schema (Fig. 8b) were used only to properly distribute load applied to the building and barrettes. The bending and torsion moments, shear and normal forces that were obtained from the analysis, although reaching large values rarely encountered in practice, allowed to design details of the reinforcement of massive concrete.

The structure was completed in 2013, and its effectiveness was confirmed by monitoring results. Downward movement of the building has slowed down to about 0.2 cm per month in 2014 (from initial speed of about 1 cm / month). In 2015 and 2016 almost no horizontal movement was observed (no more than 0.5 cm / year), as well as no damages to the building structure were observed. The designed protection of the building can be presented as a successful one. The cost of the described structural protection, was estimated as about 45% of the total investment, without stabilization.

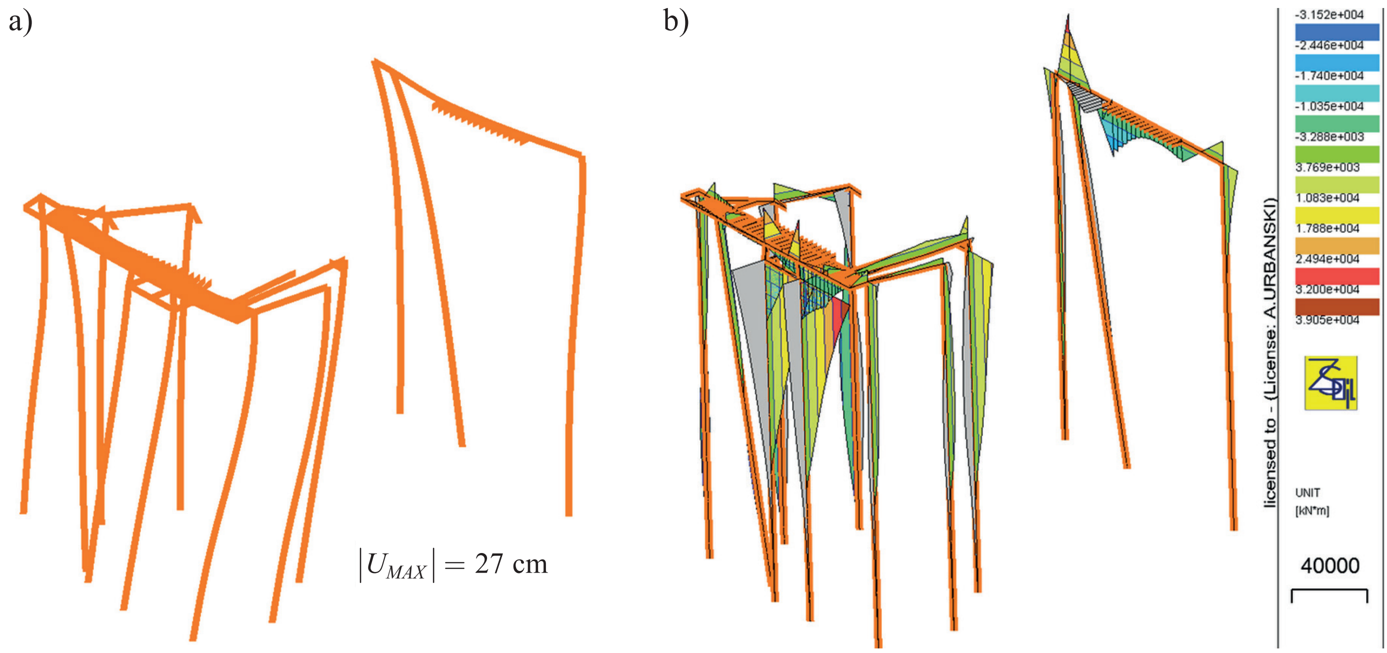

Fig. 10. a) Deformed shape of the stabilizing frame. b) Bending moments in two directions

6. Conclusions

A successful stabilization action of a building structure in an active landslide which was described in this article is an example of an interdisciplinary problem. The combined geotechnical, geodetic and structural approach supported with usage of sophisticated numerical analyses allowed to design an effective supporting structure. The structure prevents building from further movement downward along the slope. The presented design methodology can be used in similar situations, where a value of the endangered object is judged as high and a protection of the whole landslide is not possible (or uneconomical).

References

[1] D.H. Cornforth, *Landslides in Practice*, John Wiley & Sons, Inc.; 2005.

[2] D.V. Griffiths and P.A. Lane, "Slope stability analysis by finite elements", *Geotechnique*, 49, 387–403 (1999).

[3] T. Matsui and K-C. San, "Finite element slope stability analysis by shear strength reduction technique", *Soils and Foundations*, 32, 59–70 (1992).

[4] A. Ozbay and A.F. Cabalar, "FEM and LEM stability analyses of the fatal landslides at Çöllolar open-cast lignite mine in Elbistan, Turkey", *Landslides*, 12, 155–163 (2015).

[5] D. Sarah and M.R. Daryono, "Engineering geological investigation of slow moving landslide in Jahiyang Village, Salawu, Tasikmalaya Regency", *Indonesian Journal of Geology*, 7(1), 27–38 (2012).

[6] A. Truty, A. Urbański, M. Grodecki, and K. Podleś, "Computer aided models of landslides and their protection problems", *Scientific-Technical Papers of Communication Engineers and Technicians of the Republic of Poland in Krakow*, 88, 395–419 (2009), [in Polish].

[7] A. Urbański, "A simplified computational model for a periodic system of horizontally loaded piles", *3-rd International Symposium on Computational Geomechanics*, *21–23 August 2013*, Krakow, Poland (2013).

[8] M.Th. van Genuchten, "A closed form equation for predicting the hydraulic conductivity of unsaturated soils" *Soil Science Society of America Journal*, 44, 892–898 (1980).

[9] L. Wysokiński, "The methods of landslides prediction and their protection", *XXV Scientific Conference "Building failures" 2011*, 291–320 (2011) [in Polish].

[10] Y. Zheng, X. Tang, S. Zhao, C. Deng, W. Lei, "Strength reduction and step-loading finite element approaches in geotechnical engineering", *Journal of Rock Mechanics and Geotechnical Engineering*, 1(1), 21–30 (2009).

[11] Th. Zimmemann, A. Truty, A. Urbański, and K. Podleś, "Z_Soil. PC 2003 manual. Theory", Elmepress international & Zace Services ltd., Lauzanne, Switzerland, (2005).

[12] M. Berti, L. Bertello, A.R. Bernadi and G. Caputo, "Back analysis of a large lanslide in a flysh rock mass", *Landslides*, 14, 2041–2058, (2017).

[13] O. Taheri, R.Z. Moayed, and M. Nozari, "Lateral soil-pile stiffness subjected to vertical and lateral loading", *Journal of Geotechnical and Transportation Engineering*, 2015.

[14] K. Georgiadis, S. Sloan, and A. Lyamin, "Ultimate lateral pressure of two side-by-side piles in clay", *Geotechnique* 63(9), 773–745, 2013.