GOSPODARKA SUROWCAMI MINERALNYMI

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Engineering geology investigations for foundation design in extension of „Strachocina" natural gas underground storage in flysch Carpathians

Introduction

In Strachocina, near the city of Sanok depleted gas field is using as underground gas storage. By the year 2012, the Polish Oil and Gas Company plan to increase the existing storage capacity over two-times, up to the level of 330 mln m^3. Prepared civil engineering project design technical facilities and buildings foundation on the slopes built from flysch deposits. Design area, 200 m wide and 600 m long, in upper part was relatively flat, but to the north and south directions, slope inclinations increasing in some places up to 20%. On both sides of the area two deep streams valley were situated (fig. 2). Investor initially made only hand borings, but due to difficult ground conditions and shallow groundwater more detailed report and slope instrumentation were ordered. In this research, methods used by the author in Carpathians were included (Bednarczyk 2009a, b, 2008a, b, c, 2007, 2006). It covered core-impregnated boreholes, laboratory tests and preliminary slope stability analysis. Instrumentation included inclinometer, pore pressure and groundwater level measurements. Some methods from the other projects in landslide risk areas in Poland (Gil, Długosz 2006; Kotarba 1986; Margielewski 2004; Poprawa, Rączkowski 2003; Raczkowski, Mrozek 2002; Thiel 1989; Wiłun 1987; Wysokiński 1991; Zabuski, Thiel, Bober 1999) and abroad (Corominas 1996; Finlay, Mostyn, Fell 1999; Evaluation... 1970; Janbu 1998; Larsen 2002; Senneset 1998 Wysokiński 1991) were also used. Site investigation report delivered new geotechnical data. Area had complicated geology and tectonic structure. Upper part was built from stiff

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Fig. 1. Design investment area

Rys. 1. Teren projektowanej inwestycji

shale's and claystons, inclinated vertically. Slopes included weathered claystons, shale's and clayey soils with low geotechnical parameters, highly saturated to the depth of 11–16 m bellow the natural terrain level.

1. Geology

Research area was localized inside the Strachocina anticline, in the eastern part of Central Carpathian Depression (between San and Wisłoka River Valleys). Flysch type shale's mudstones and sandstones represented marine soils and rocks from Paleogene period with thickness of 365 m (Archiwalne materiały... 1990; Przewodnik... 1973). These deposits, elevated and folded during Alpine orogenesis, formed nappe type structure. Older, relatively stiff shale's from Eocene (hieroglifowe layers) were localized in the middle of the structure and younger claystones, clays and sandstones from Oligocene (melinitowe layers) on both sides. Stratification represented steep layers inclinations on its borders and more gentle but various at the top. Main NW-SE faults (along the anticline) and secondary SW-NE, transverse faults cut the antycline. Natural terrain relief was partly from Tertiary and Quaternary periods. Erosion and mass movements processes were characterized by thick weathering zones, which formed two deep valleys, with depth of 3–16 m, on the borders of investigated area. Lithology were represented by gravely loams, silty loams with gravel and weathered fractured rocks. Many thin layers of flysch deposits were covered in upper part by weathering zones. Slope morphology were formed partly by erosion and landslide geodynamic processes.

2. Engineering geology investigations

Inside the site engineering geology report drillings, laboratory tests, geophysical GPR profiling, ground movement and pore pressure monitoring were performed (fig. 2).

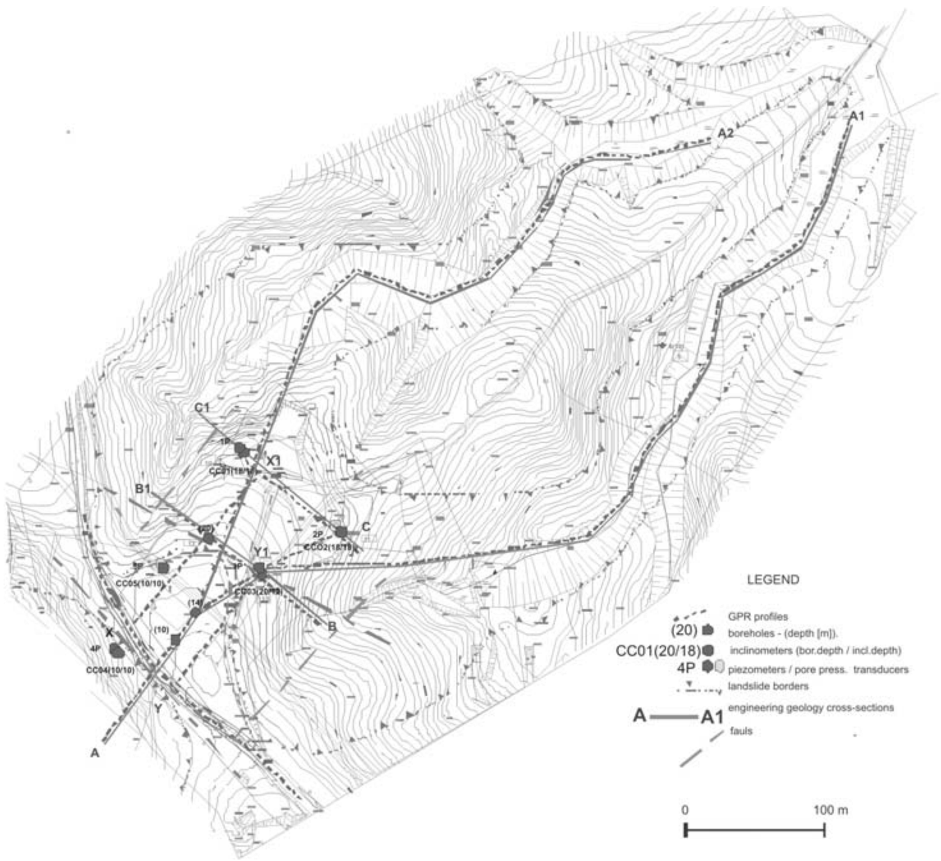

Fig. 2. Site documentation map

Rys. 2. Mapa dokumentacyjna

2.1. Core drillings

Twelve core impregnated drillings, diameter of 132 mm and length of 150 m, were performed. It included detailed soil description and field shear strength tests. Fifty undisturbed soil samples were taken for the laboratory tests. In boreholes, six inclinometers to the depth of 10 to 18 m together with pore pressure transducers and Casagrande piezometers were installed.

2.2. Geophysical investigations

Eight profiles, totally 2130 meters of Ground Penetration Radar (GPR) scanning with 100 MHz unshielded antennas were included in engineering geology site investigations. GPR was performed for detailed recognition of geological stratification along the engineering geology cross-sections. It allowed for precise recognition of stratification and tectonic structures to the depth of 15–20 m. Calibrations of GPR results were made using boreholes. For different types of rocks and soils (stiff shale's, mudstones, claystones and colluviums) characteristic values of dielectric constant, attenuation (dBm^{-1}) permittivity and permeability values were included (Dunicliff 1988). Ground Vision software was used for data interpretation. Morphology were included using CorelDraw software. Visualization of GPR results included filtration of scanning profiles using colors. For better visualization on the cross sections yellow color indicated mudstones, grey claystones (fig. 3). Interpretation included also indication of faults and sliding surfaces. GPR detected that colluviums were localized to the depth of 7–9 m. It well correspond with heavily saturated colluviums detected in the boreholes. GPR profiles delivered tectonic, stratification and colluviums depth data. However it is necessary to conclude that it was possible with correlation by detailed site investigation and monitoring results. In GPR method layer depths are only interpretation result and should be calibrated very carefully.

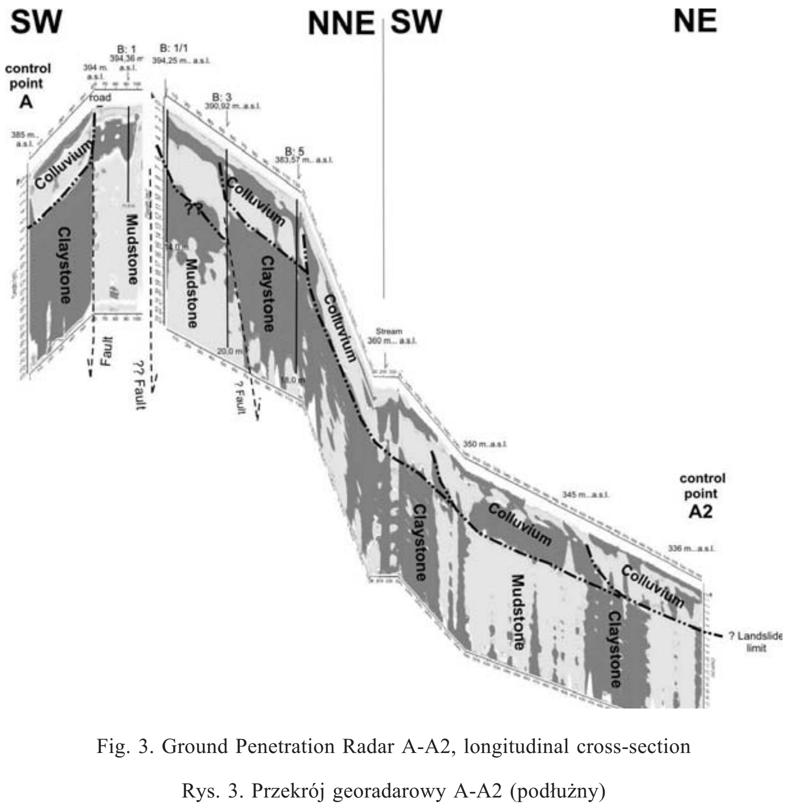

Fig. 3. Ground Penetration Radar A-A2, longitudinal cross-section

Rys. 3. Przekrój georadarowy A-A2 (podłużny)

2.3. Instrumentation and monitoring

Instrumentation for potential active ground movements recognition included installation of inclinometrs to the depth of 18 m. It also covered devices for groundwater level and pore pressure measurements. Monitoring system was consist of four monitoring places (instalation May–July 2008). At the and of 2008 instrumentation were extended by two additional points. Totally, 68 m of ABS, 70 mm, inclinometer casings, two pore pressure transducers and Casagrande piezometers with filter at the depth of 5 m were installed. Biaxial UK produced inclinometer probe were used for measurements. Data were logged every 0.5 m with accuracy varied from 0.1 to 0.3 mm in two directions – A (parallel to the ground movement) and B (transverse). Measurements were repeated for false data elimination (probe were rotated by 180 degree). Casings inclinations were transverse from degrees into lateral displacements using trigonometric formulas (Corominas 1996). It allowed detection of movements, its directions and depths. Displacements in every borehole were calculated by comparison of actual and initial measurements, using equation:

$$\Delta u_{iy} = Lx[\sin(\theta_{iy}) - \sin(\theta_{io})] \tag{1}$$

where:

θ_{iy} – probe inclination angle in (i) section and (y) series,

θ_{io} – probe inclination angle in (i) section in reference series.

Borehole cumulated displacement was calculated as a sum of all above increments from bottom to the top using formula:

$$\sum \Delta u_{iy} = \sum \{Lx[\sin(\theta y_{iy}) - \sin(\theta_{io})]\} \tag{2}$$

Depending on localization, three to ten ground movement measurements were performed till the end of February 2009. At lower part of design area (fig. 4) after nine measurements (May 2008–February 2009) cumulative displacements in A direction reached values 4.0–7.1 mm to the depth of 16m with some changes within this depth. The largest displacements were registered to the depth of 11 m.

In the central part of design area after six measurements (July 2008–February 2009) cumulative displacements to the depth of 16m reached value of 3.0–5.4 mm (fig. 5). Displacements were changing at the different and indicated that colluviums were not homogenous. South from the design area, however only three measurements were performed cumulative displacements of 2.2 mm to the depth of 1.5 m was observed. At the upper part of design project area characterized by good engineering geology conditions ground movements were no detected.

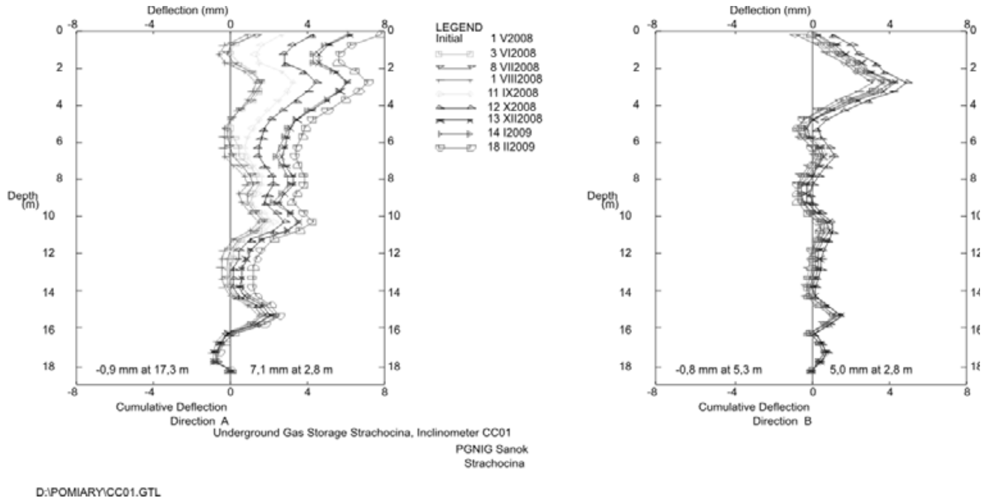

Fig. 4. Cumulated displacements inclinometer CC01 – lower part of research area

Rys. 4. Skumulowane przemieszczenia inklinometr CC01 – dolna część terenu

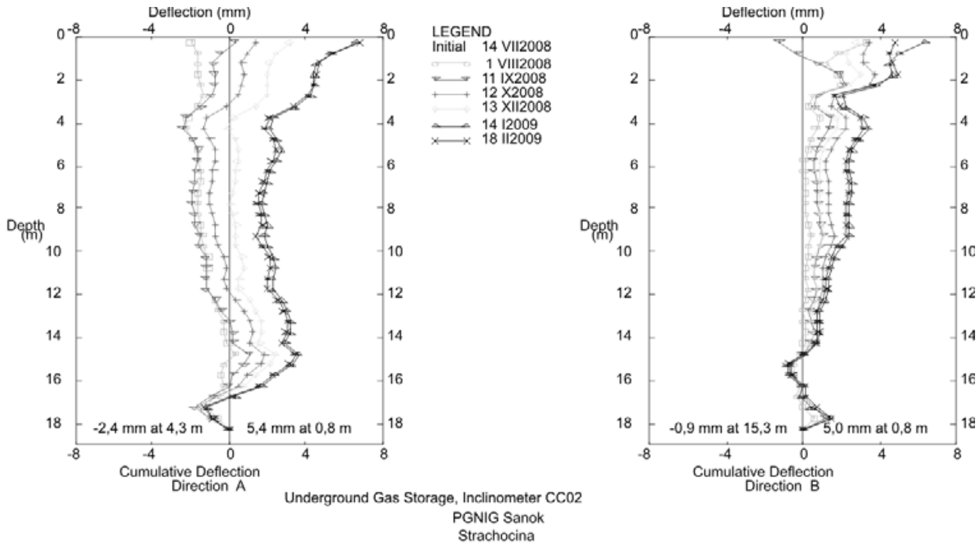

Fig. 5. Cumulated displacements inclinometer CC02 – central part of research area

Rys. 5. Skumulowane przemieszczenia inklinometr CC02 – środkowa część terenu

TABLE 1

Ground movements monitoring results

TABELA 1

Wyniki monitoringu przemieszczeń wgłębnych

No.	Displacements/depth [mm/m] direction A	Displacements/depth [mm/m] direction B	Number of measurements	Inclinometer
1.	7.1/12 (16)	5.0/12 (16)	9	CC01
2.	5.4/16	5.0/16	7	CC02
3.	5.8/16	6.0/11	6	CC03
4.	2.2/1.5	1.4/1.5	3	CC04
5.	0.0 (+-1.0)	0.0 (+-1.0)	2	CC05

TABLE 2

Pore pressure monitoring results

TABELA 2

Wyniki monitoringu ciśnienia porowego

No.	Date	Pore Pressure CC02 [kPa]		Pore Pressure CC05 [kPa]	
		Method I	Method II	Method I	Method II
1.	02-06-08	37.57	33.64		
2.	10-07-08	41.77	32.64		
3.	10-09-08	29.03	22.86		
4.	15-10-08	21.64	15.27		
5.	12-12-08	29.49	25.43		
6.	19-01-09	27.11	10.71	44.28	39.25
7.	20-02-09	27.21	21.38	39.92	31.42

2.4. Laboratory tests

Fifty sets of index tests were performed (grain size, moisture content, unit weight, bulk density, density of solid particles, Attenberg limits, and organic material content). Also shear strength, consolidation and borehole groundwater chemical tests were made. In direct shear box apparatus seven tests with shearing speed of 0.1 mm/min were realized. Using four incrementally loaded oedometer consolidation tests oedometer modulus were predicted. Obtained results showed very high, up to 51%, moisture content (mainly 25–30%). Liquidity index varied significantly from the stiff to soft clays with high plasticity (IL = 0.56–0.78 in colluviums to the depth of 3 m). Clays, silty loams, gravels and loamy gravels were indicated as a main kinds of soils. Content of organic/bituminous material were high, in some cases

2–10%. Shear tests, however gave probably overestimated results of friction apparent angle and cohesion. It was caused by high content of claystones (crushed rock particles) inside the clays, which were blocking boxes of shear apparatus. For slope stability analysis following parameters of colluviums were included $\varphi_u^{(n)} = 7^o$, $c_u^{(n)}$ =20 kPa, ρ = 1.863 g/cm^3 (IL = 0.56). Consolidation tests showed that colluviums clayey soils are extremely compressive to the depth of 11 m, what suppose high subsidence under external loads. For colluvium soils oedometer modulus were: $E_o^{(n)}$ = 220 kPa, $M_o^{(n)}$ = 11000 kPa. In tests incremental loads of σ_n = 12.5, 25, 50, 100, 200 i 400 kPa were used. Very long, up to two weeks time consolidation of soil samples were observed. Laboratory tests results shows that soils with the highest moisture content (30–50%) and plasticity index were localized to the depths of 10 m (fig. 8).

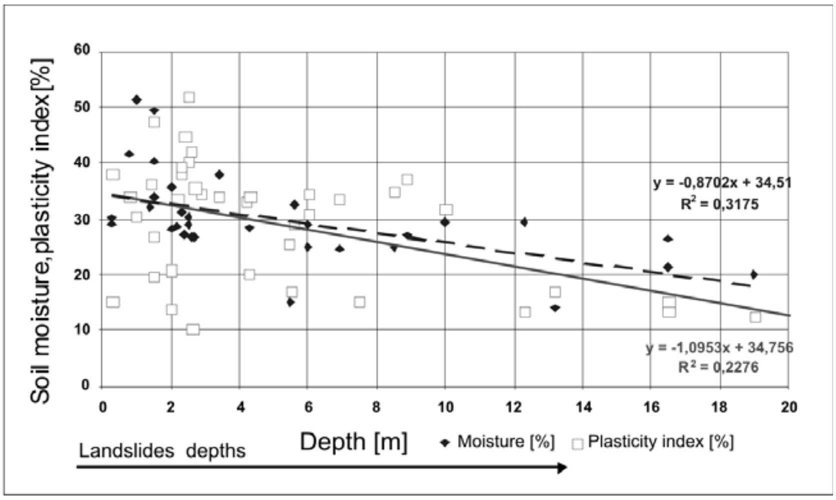

Fig. 6. Results of moisture content and plasticity tests index with depth

Rys. 6. Wyniki badań wilgotności i wskaźnika plastyczności gruntów wraz z głębokością

3. Groundwater and climate conditions

Shallow groundwater level, varied between 0.5 and 1.5 m was observed. In loamy slope weathering zones it depend on slope inclination and precipitation values. At lover parts of slopes and swampy areas was equal to the surface level. High precipitation values and thick weathering zones had visible influence on high pore pressure values. Groundwater seepage decreased shear strength on the slopes. Pneumatic pore pressure transducer installed at the depth of 5 m indicated high pore pressure values of 29 to 45 kPa. It increased nearly two times after high rainfalls in July 2008. Yearly precipitation values in Pakoszoka village near to this area reached volume of 912 mm. Climate is characterized by intensive rainfalls delivering large amounts of waters on the slopes. Natural moisture contents in soils localized on upper part of the terrain were very high and reached values of 35–50%.

4. Engineering geology conditions

Engineering geology conditions were dependent of soil types, slope inclinations, occurrence of faults and discontinuities, vertical layer inclination (prone for groundwater infiltration) and shallow groundwater level. Soils had different strength parameters and digenesis levels. Typical soils were represented by loamy gravels, clays, loams with gravel, claystones, and thin layers of sandstones. Upper flat part of design project area had relatively good soil conditions with stiff mudstones and claystones inclinated vertically. North to this area, on the other site of fault pararel do anticline axis, highly saturated with high plasticity colluviums and weathering zones are localized (especially close to the valleys). To the south, near the public road, another landslide area were localized. Boreholes and GPR scanning were the basis for four engineering geology cross-section construction. Longitudinal cross-sections A-A1 and A-A2 were localized along the design project area to the valleys in north directions. Transverse cross-sections B-B1 and C-C1 were perpendicular to it. (fig. 2, 3, 4, 8).

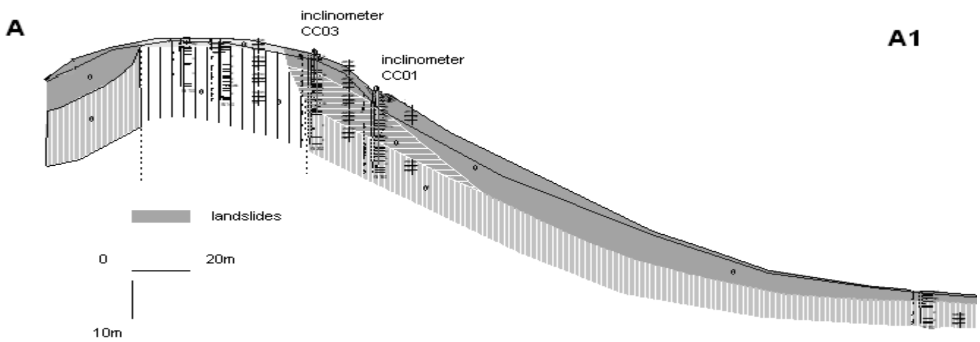

Fig. 7. Engineering geology cross-section A-A1

Rys. 7. Przekrój geologiczno-inżynierski A-A1

Sections illustrated geology and tectonic structure of investigated part of Strachocina Anticline well. It was detected that landslide zone to north is consist of two parts moving into valleys divided by the bedrock uplift. Valleys were formed probably in the places where transverse to the anticline faults occurred. Boreholes detected that colluviums were localized to the depth of 5–16 m and consist of two types of soils. Upper represented clayey cohesive soils with low geotechnical parameters and very high moisture content. Lower crushed claystones and rock boulders. Preliminary slope stability analysis by LEM Janbu method (Margielewski 2004) indicated that relative factor of safety were very low, $Fs = 0.87$ (when $Fs \geq 1.3$ slope is in stable conditions). The most probable slip surface was localized in the upper part of the slope in foundation area. These analysis will be extended in other locations and checked by FEM analysis, however obtained results indicated that part of the design area have to be build on piles foundations.

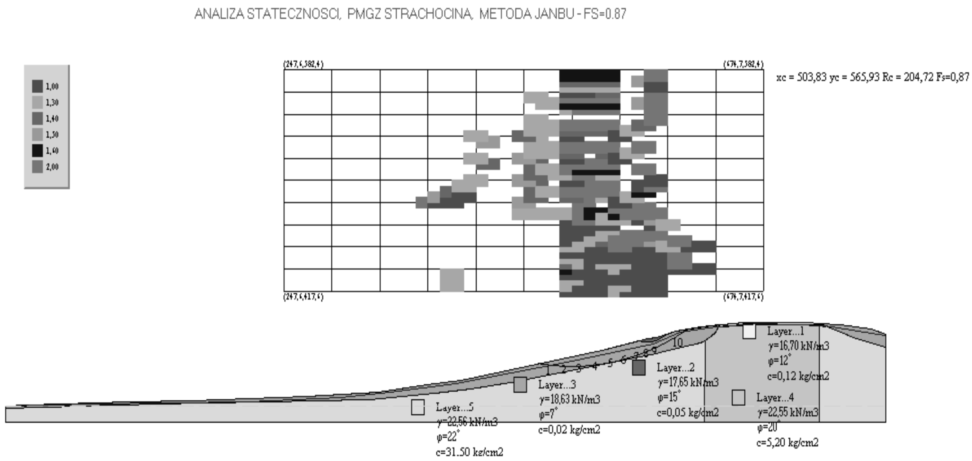

Fig. 8. Slope stability analysis by Janbu method

Rys. 8. Analiza stateczności metodą Janbu

Conclusions

Presented investigation detected that some parts of the terrain for underground gas storage „Strachocina" are localized in active ground movement area. Colluviums to the depth of 2–16 m, shallow groundwater level, low geotechnical parameters of clayey soils with high plasticity. It excluded part of area for standard foundation design. Monitoring measurements after 3–9 tests which started from December 2008 detected movements of 2–7 mm. Area was characterized by complicated geologically and tectonically structures, vertical layers inclinations prone for water infiltration, highest values of groundwater pore pressure and low slope stability conditions. Hopefully upper part of the terrain was characterized by good soil conditions with shallow bedrock depth (stiff mudstones, claystones and sandstones). Investigation proved that it is necessary to change localization of infrastructure including some buildings and compressors in safe place and others including gas supply pipes should be built on piles foundations. Internal and surface drainage systems, and some changes in the project will be also necessary. It should include withdraw from plans to built ponds and embankments in the lower part of the design area. These plans are possible in the design stage time and will be included in corrected version of the design project. Changes in infrastructure localizations and piles foundations for some others facilities have to safeguard gas storage infrastructure. It should be also check by control monitoring measurements and slope stability analysis.

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ENGINEERING GEOLOGY INVESTIGATIONS FOR FOUNDATION DESIGN IN EXTENSION OF „STRACHOCINA" NATURAL GAS UNDERGROUND STORAGE IN FLYSCH CARPATHIANS

Key words

Engineering geology, geotechnical monitoring, *in-situ* tests, laboratory tests

Abstract

In the paper results of soil investigations for design of underground gas storage infrastructure foundations are presented. By the year 2012 Polish Oil and Gas Company will increase storage capacity of 150 mln m^3, over two times. It will be necessary to make foundation of compressors, gas cold storages and buildings on slopes built from fysch deposits. They are characterized by different strength parameters and complicated tectonically. Investigations included core drillings, undisturbed sampling, ground penetration radar (GPR) scanning, geotechnical laboratory index, consolidation, oedometer IL and direct shear-box tests. Different test methods, including ground movements inclinometer and piezometer measurements together with slope stability analysis allowed soils characterization. Significant differences between flat area localized on top with shallow depth of stiff rocks and colluviums valleys to the north and south were detected. Investigations proved that foundation of infrastructure will be possible, but it requires changing in design project including some buildings and other elements of infrastructure. For some parts of slopes piles foundations have to be constructed. Also control monitoring measurements will be necessary.

BADANIA GEOLOGICZNO-INŻYNIERSKIE DLA ROZBUDOWY PODZIEMNYCH MAGAZYNÓW GAZU ZIEMNEGO „STRACHOCINA" W KARPATACH FLISZOWYCH

Słowa kluczowe

Geologia inżynierska, monitoring geotechniczny, testy *in-situ*, testy laboratoryjne

Streszczenie

W artykule przedstawiono wyniki badań podłoża gruntowego dla celów posadowienia obiektów infrastruktury związanych z podziemnym magazynowaniem gazu. Do roku 2012 planuje się zwiększyć pojemność magazynów Strachocina ponad dwukrotnie, o 150 mln m^3. Niezbędne jest w tym celu posadowienie sprężarek,

chłodni gazu i innych budynków na stoku antykliny fliszowej, cechującej się zmiennymi parametrami wytrzymałościowymi i skomplikowaną budową tektoniczną. W celu rozpoznania budowy podłoża wykonano wiercenia rdzeniowe, opróbowanie NNS, profilowania GPR, badania podstawowych parametrów fizycznych gruntów, edometryczne badania ściśliwości oraz badania parametrów wytrzymałościowych gruntów w aparacie bezpośredniego ścinania. Zastosowanie różnorodnych metod badawczych, monitoringu inklinometrycznego i piezometrycznego oraz modelowanie stateczności pozwoliło na charakterystykę podłoża gruntowego. Stwierdzono wyraźną różnicę pomiędzy płaskim obszarem zlokalizowanym na szczycie antykliny, charakteryzującym się płytkim zaleganiem skał zwięzłych a dolinami na północ i południe od niego, gdzie wykryto przemieszczenia wgłębne. Posadowienie projektowanej infrastruktury, w taki sposób, aby zapewnić jej maksymalne bezpieczeństwo, jest możliwe. Wymagać to będzie jednak zmian w projekcie, przeniesienia części obiektów oraz posadowienia pozostałych na fundamentach palowych. Niezbędne będzie także prowadzenie monitoringu w celu kontroli wykonanych zabezpieczeń.