INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Ground treatment and stabilization of a railway embankment in failure state

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1 Introduction

The E-65/C-E65 railway line runs from Gdynia on the coast to Warsaw (see Fig. 1). The magisterial line E65 is an important part of the 6th Trans-European transport network connecting the Baltic Sea countries with the Adriatic Sea countries and the Balkan region. The rail line through Jurkowice, where the case study is situated, was originally built as a single track in the mid-19th century and was converted into a double track with embankment widening in 1967 (Lijewski 1977). Few years ago, Polish State Railways undertook a major network upgrade to create a high speed rail system, capable of handling modern Pendolino trains with a top speed of 200 km/h.

Fig. 1: Railway line E65/C-E65 from Gdynia to Warsaw with highlighted location of Jurkowice (embankment in failure state)

2 Geological conditions

Ground investigation on the site, before planned works, revealed non-bearing soils up to 10 m depth under the left side and up to 17 m under the right side of the embankment (see Fig. 2), forming a natural 10^{o} dip of sensitive layers, which corresponded with the direction of future landslide. The thickness of the organic soils – especially muds and peat – suggested that the embankment was built in an old river bed. Beneath the layer of organic soils (stratum IIa) there was an approximately 5 m thick layer of soft and firm clays (stratum IV and V). Several decades of overloading the ground with the 8 – 10 m high embankment resulted in the creation of a natural basin from anthropogenic soils and a partial displacement of underneath organic soils. The thickness of organic layers underlying the embankment, determined in field tests, reached up to 4 m. The process of consolidation had a significant impact on the strength parameters of the soils located beneath the embankment, which were increased comparing to the characteristics of layers situated outside the embankment. The undrained shear strength of the organic layer ranged from 10 to 35 kPa, but locally the strength was as low as 5 kPa.

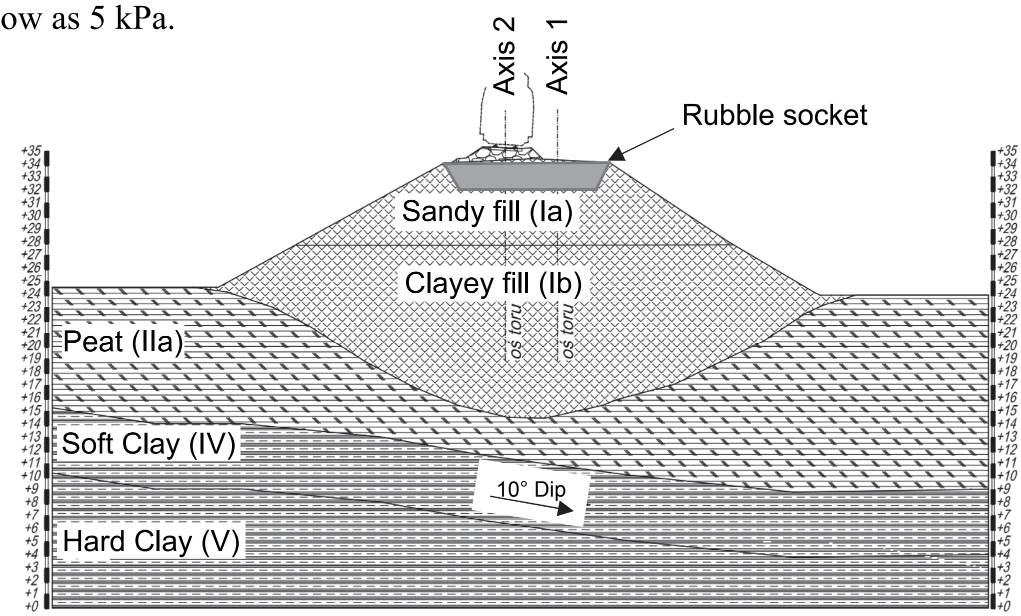

Fig. 2: Typical cross – section of the embankment in critical chainage 266+300 (initial state, before the landslide)

3 Failure mechanism

The early stability issue was noticed on site, following the modernization works, which included a 1 m embankment widening on its right slope (see Fig. 3a). Before placing a protective layer in the axis of track no. 1, the top of the embankment subsided, showing a 50 mm crack (see Fig. 3b). The observable crack took the shape of a 130 m long curve, from chainage 266+210 to 266+340 (see Fig. 4), with settlements of up to 300 mm.

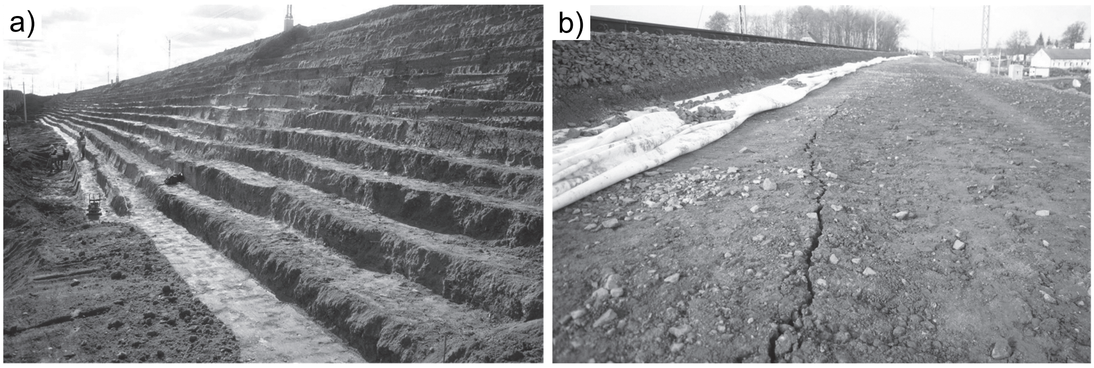

Fig. 3: a) Shelves formed to widen the embankment 1 m to the right side
b) Crack on the top of embankment under the track no.1

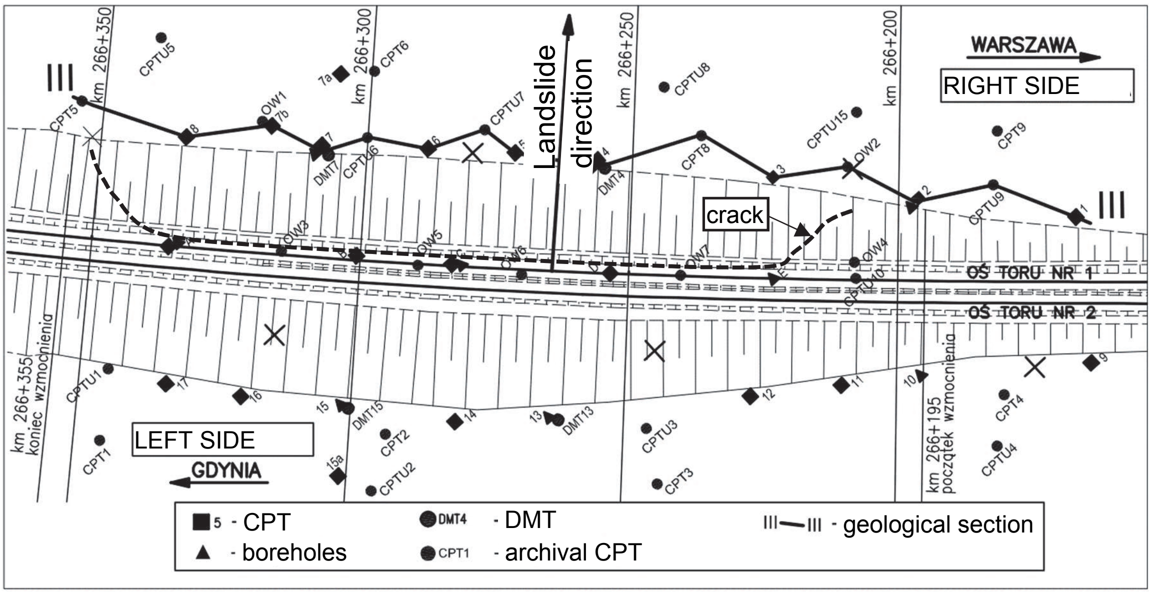

Fig. 4: Layout of the field tests and the observed crack curve on the right side of the embankment

The geometry and character of instable slope indicated that the embankment was failing and had been set in motion as a result of earlier formed deep-seated shear failure. The back analysis of this state proved that the overall stability of the embankment was unsafe (see Fig. 5).

It should be mentioned that during works the 3 m layer of rubble pressed into the embankment was found, suggesting that running track was regularly underpinned due to progression of embankment's settlement over the years (see Fig. 2).

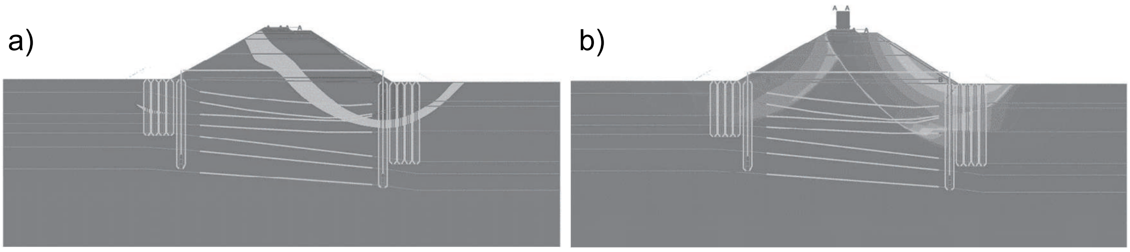

Fig. 5: a) Stability for the initial state without soil improvement, FOS=1.1 without traffic load applied
b) Stability with long term traffic load applied on a single track, FOS≈1.0

According to fact that the constantly monitored track no. 2 didn't experience any displacement, it was decided to remain it open to passenger services with a speed limit of 10 km/h and close the track no. 1 only.

4 Initial design solution

In order to provide slope stability during the main repair works, the construction of a soil buttresses on both sides of the embankment was performed at first. The soil buttresses were used as working platforms for specialist machinery used on site. Due to very low soil strength parameters at the bottom of the embankment, it was decided to strengthen the buttresses with concrete columns. Then a CFA pile wall with a diameter of 80 cm in axial spacing of 1 m was constructed. The structure wall, embedded in the bearing layer at necessary depth, ensured unconstrained flow of groundwater, preventing the problems connected with water damming. In the next phase of works, it has been planned to top the pile walls with capping beams and to tie them up with post-tensioning system strands, performed in leading pipes, installed in horizontal drillings at the embankment's foundation. In the last stage of repair works, the execution of compaction grouting was planned in the embankment and its underlying layers of low bearing soils, providing a deep compaction of the soil that had been loosened during the creation of the landslide.

The stable construction, performed in that way, cuts through a deeply developed slip surface, preventing the further vertical movement of the embankment during its exploitation. It additionally prevents from the appearance of potential landslides on the other side of the embankment, which is in a state of unstable balance FOS≈1.0, according to calculations (see Fig. 5b).

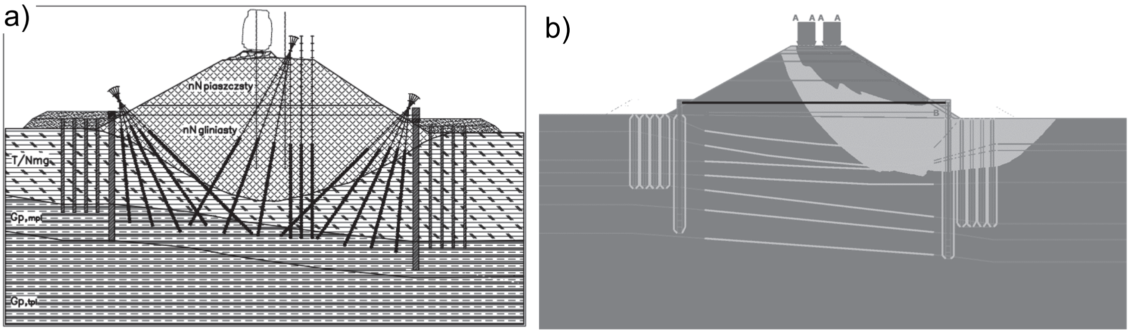

Fig. 6: a) The first geotechnical solution
b) Stability with soil improvement, FOS_{min}=1.5 with a long term traffic load applied on a double track

Once the stabilization design was finished and the work was already planned, torrential downpour activated the landslide and caused the embankment to fail. During the time of rain the slope became unstable with the reduction of the soil suction. Due to rain infiltration, suction pressure in the soil decreased, and thus, the effective shear strength of a slope significantly reduced, resulting in reduction of the factor of safety (FOS). The potential mechanism of rain-triggered slope failure of the railway embankment was widely discussed and reported by Raj and Sengupta (2014).

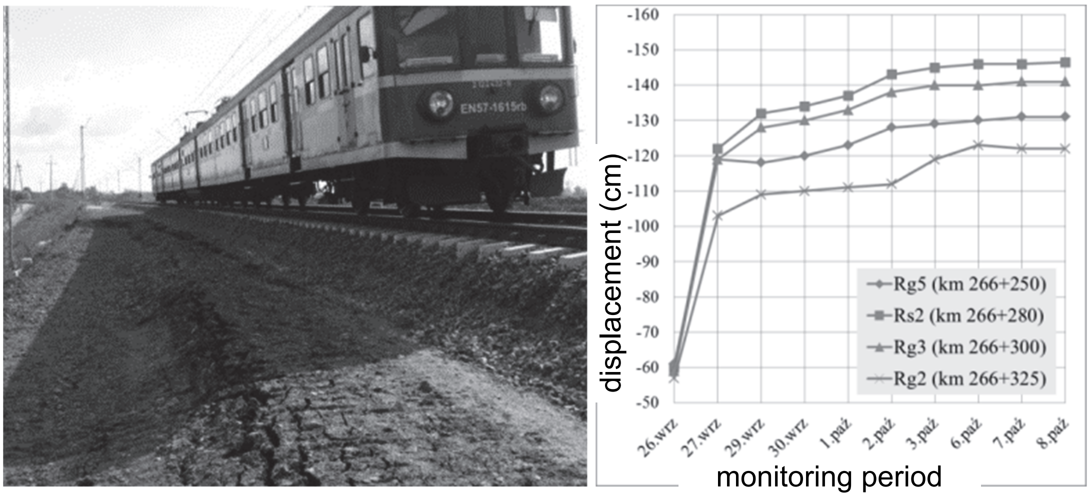

Fig. 7: a) Top right side of the embankment (~60 cm of settlement) – day after downpour
b) Time – displacement curves of active landslide

The right side of the embankment was immediately covered with a PVC foil, preventing from further destruction that could be caused by changeable weather conditions. Additionally, 6 inclinometers were installed at the bottom right side of the embankment to measure the horizontal deformation, and precise geodetic survey was done. In order to estimate the compaction state of existing embankment on the landslide side, a number of dynamic penetration tests were performed. As shown on Fig. 7b, displacement of the landslide was up to 1.5 m in two weeks and began to stay constant after this time. It suggested that the landslide was stabilized and repair works can be resumed.

Both dynamic probings and type of deformation clearly indicated that the loosened mass of the soil was an extension of the embankment made in 60's of the last century (Lijewski 1977). The widening on the right side added another track to the former single track railway.

The only right way to fulfill the safety requirements and criteria of the modernized railway embankments was to disrupt the railway traffic. As closing a line of such importance is unfortunate, the repair works were limited to maximum 32 days.

5 Implemented design solution

As a result of detailed analysis of the critical situation, a new solution was proposed. It assumed an adaptation of the design solution to actual conditions, providing a comparable safety factor and making it possible to finish works on time. Revision included track no. 2 disassembly and a partial embankment removal. The compaction grouting columns from the right side and from the top were replaced with Controlled Stiffness Columns Φ40 cm (CSC) with load transfer platform (LTP) on top (see Fig. 8).

After the completion of CSC and LTP, inclinometers were installed in order to control the vertical displacement in the bottom of embankment. Afterwards, the construction of the next layers of the embankment was performed. The connection between the existing embankment and new layers was constructed as a slope with 60x80 cm steps. To unify the parameters of the top, the last meter of the embankment was constructed as one joint layer for the new and existing part.

The acceptance criteria for the railway bed were set according to Polish State Railways' ID-3 recommendations (2009), to the depth of 2 m from the top of the protective layer – compaction index $I_s \geq 1.00$, below 2 m from the top of the protective layer $I_s \geq 0.95$.

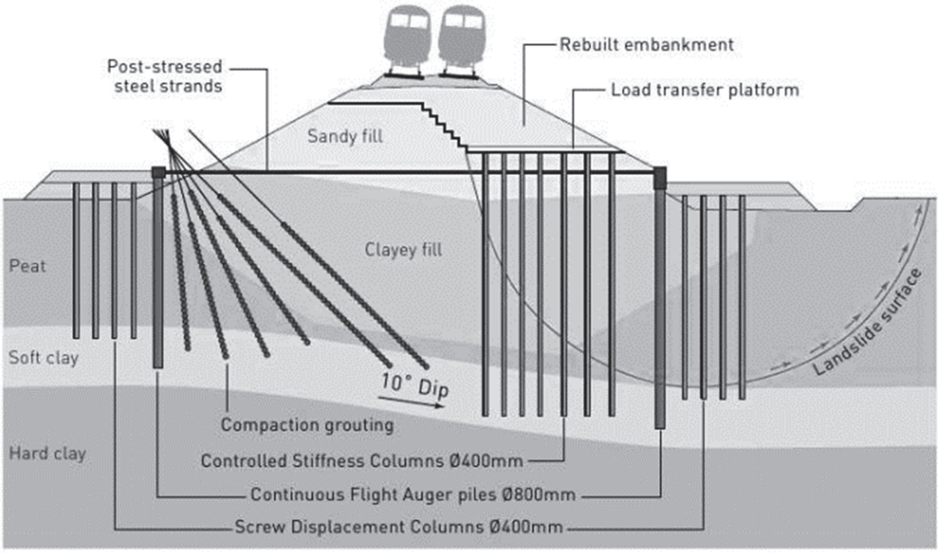

Fig. 8: Implemented design solution

6 Conclusions

The example concerned the execution of a complex embankment improvement in failure state in complicated ground conditions, under strict time pressure. Furthermore, limited working space was a real challenge for the site managers, coordinating the workers and several heavy machines on site. An efficient management of the construction site resulted in a successful termination of the project, without any accidents.

After the completion of the repairing works, regularly performed Quality Control results (measuring vertical and horizontal displacements) indicated stabilization of the settlements, which proves the adopted solution to be safe and effective (see Fig. 9) (Mitrosz 2015).

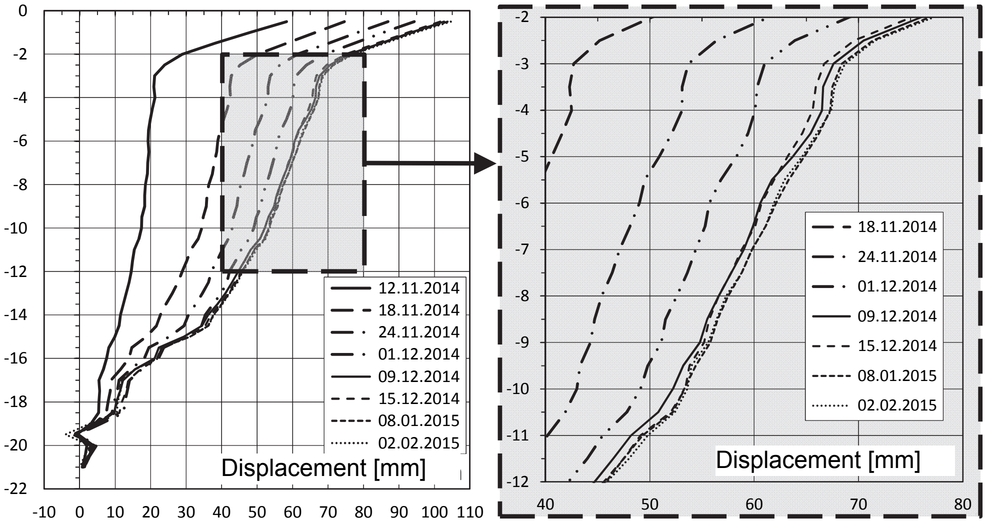

Fig. 9: Horizontal displacements of the embankment measured by inclinometers. Base measurement performed on 12.11.2014 (after finishing CSC), reference measurement on 15.12.2014 (end of strands stressing)

Such a complicated and problematic task brings a valuable lesson, that the power of nature should never be underestimated during construction works. Moreover, it is crucial to assure security of the workers and construction.

Fig. 10: Rebuild embankment on landslide side

7 Literature

Lijewski T. (1977)
Geografia Transportu Polski, Państwowe Wydawnictwo Ekonomiczne, Warszawa.

Mitrosz O. (2015)
Wzmocnienie nasypu kolejowego w stanie awaryjnym w ciągu linii E65 w miejscowości Jurkowice. XXVII Konferencja Naukowo-Techniczna Awarie Budowlane 2015.

PKP PLK S.A. (2009)
Warunki techniczne utrzymania podtorza kolejowego Id-3, Warszawa.

Raj, M. & Sengupta, A. (2014)
Rain - triggered slope failure of the railway embankment at Malda, India, Acta Geotechnica (2014) 9: 789-798.