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INVESTIGATION OF LANDSLIDES AFFECTING A ROMANIAN RAILWAY

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ABSTRACT

The study of the landslides that occurred on the hillside between Covurlui plateau and Brates Lake is presented, together with geotechnical investigations performed in order to establish the trigger mechanism of the registered cyclical instability phenomena. Laboratory test results showing the peculiarities of the clay strata, mineralogical, physical, and mechanical properties are discussed, and stability analysis results are commented, in order to substantiate the proposed consolidation measures.

KEYWORDS

Landslide, Railway, In situ investigations, Stability analysis, Earth thrust.

INTRODUCTION

The railway Galati-Barlad is located, between km. 2+200 and km. 16+500, on the border of the Brates Lake, at the bottom of a cliff having height of about 60m. Put into service between 1903 and 1912, the railway represents the main link between town Galati and the localities in the north of the Moldavia, assuring both the traveler's transport daily traffic of 5000-6000 passengers and goods, especially food products, from and to Galati.

After 1970 the already existing landslide process became more and more extended, with periodical development, affecting the railway traffic between Galati (km. 2+200) and Tulucesti station (km. 12+500).

The occurrence of the vast landslides coincided always with spring thaw periods or intensive and lasting rainfalls. Soil displacements started for first stage of instability (Fig. 1), by the detachment of a plot of land, from the border of the plateau Covurlui (Fig. 2) and pushing of sided masses, downwards.
The cyclical character of landslides is illustrated by the following data (Table 1).

Table 1. Characteristic Data of the Landslides.

<table>
<thead>
<tr>
<th>Year</th>
<th>Affected Sections of Railway</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1974</td>
<td>km. 2+400 to 2+700; km. 5+000 to 5+540</td>
<td>0.840</td>
</tr>
<tr>
<td>1976</td>
<td>km. 5+100 to 5+800; km. 7+180 to 7+380</td>
<td>0.900</td>
</tr>
<tr>
<td>1980</td>
<td>km. 5+500 to 8+550</td>
<td>3.050</td>
</tr>
<tr>
<td>1985</td>
<td>km. 5+900 to 9+350</td>
<td>3.450</td>
</tr>
<tr>
<td>1991</td>
<td>km. 5+000 to 6+200</td>
<td>1.200</td>
</tr>
<tr>
<td>1992</td>
<td>km. 2+200 to 2+800; km. 5+700 to 6+500</td>
<td>1.400</td>
</tr>
<tr>
<td>1993</td>
<td>km. 5+500 to 7+800</td>
<td>2.300</td>
</tr>
</tbody>
</table>

In April 1993, after the thaw period, a big landslide began to develop by the detachment of a plot of land of about 5 to 10m width from the upper platform, on a length of about 2.300km. As shown on figures 3 and 4, the railway had horizontal maximum displacements of 20 to 25m and vertical movements of about 10 to 15m.

Due to important soil mass displacements, the plantations in affected zone were destroyed and a section of an irrigation canal, together with a tributary of the Brates Lake, were dislocated and interrupted.

In order to design the necessary consolidation measures, and to re-establish normal running conditions for the affected railway, topographical, geotechnical, geophysical, and hydrological studies were performed.

Taking into account the total length of the sections with active landslides, of about 14km, a trial zone was chosen in the unstable zone, between km 6+050 and km 6+250, on which the proposed consolidation measures were to be checked and homologated, and afterwards extended on the entire railway portion affected by landslides.

IN SITU INVESTIGATIONS

The layout of the trial zone is presented on Fig. 5 where the geotechnical and hydrological boreholes together with inclinometric measurements performed to establish the stratigraphy together with hydrological conditions of the site, are located.

The borehole B6 on the Covurlui plateau in a stable zone highlights the presence of two main geological formations, as follows:

a) Loess soil strata to a depth of about 25 to 30m, with loamy interbedded horizons.

b) Structured silty clay named Tulucesti clay, after the synonym nearest locality.

The free water table in the stable zone was found at the bottom of the loess formation, where the soil is saturated within a thickness of about 5m.
A sand layer with maximum thickness of about 5m is found under the clay formation in the boreholes from the slope toe, B1 and B2 (Fig. 6) at the Brates Lake border.

In order to determine the slip surfaces position, resistivity logs were performed. On Fig. 7 the results of these field tests by means of equal resistivity values, are presented. One can observe the separation curve between the sliding and the stable mass.

Besides the information obtained by the geological boreholes and resistivity log investigations, inclinometric measurements, in boreholes equipped with flexible tubes, were performed. Based on these measurements the thickness of the sliding mass resulted of about 28 to 32m in boreholes B4 and B3 and 10 to 12m in boreholes B1 and B2 (Fig. 6).

Corroborating the configuration of the ground surface for two significant years in which the instability phenomena reactivated (1985 and 1993), with the results of the geotechnical field investigations, the appropriate sliding surfaces are plotted in Fig. 6. One can observe that the moving mass is divided by a lot of sliding surfaces of different lengths, crossing the Tulucesti clay stratum.

In the triggering mechanism suggested by Figs. 1 and 2, the detached plot from the border of the Covurlui plateau pushes progressively the soil volume previously accumulated on the slope. Short shear surfaces as T3 (Fig. 6) are producing upward movements of the unstable mass, with a bulging zone on the slope, between boreholes AB6 and B4, while long shear surfaces, as T4, are causing dominant horizontal displacements of the soil at the slope toe. Taking into account short shear surfaces as T10 acting at the lake border side, it might be possible that the afferent soil volume moves with higher speed than that of the upper slide mass. In the unstable sector, the landslide has a delapsive character, opposite to the detrusive one, of the upstream moving soil. Thus, depending on the stage of the instability process, different portions of the slid mass are moving with variable velocity, standstill periods being possible for the whole displaced mass or for portions of it.

**LEGEND:**
- B1, B2: GEOTECHNICAL BOREHOLE
- IM: INCLINOMETRIC MEASUREMENTS
- T1, T4: FAILURE SURFACE

Fig. 6. Geotechnical field investigation and probable slip surfaces.

**Fig. 7. Resistivity log results.**

GEOTECHNICAL LABORATORY INVESTIGATIONS

In order to determine the physical and mechanical characteristics of the Tulucesti clays, laboratory tests were carried out.

First of all a chemical-mineralogical analysis was performed. From the mineralogical point of view the studied clays are containing montmorillonite (30 to 38%), illite (48 to 89%), and Kaolinite (11 to 31%). The differential thermal analysis, which carried out, revealed the montmorillonite saturation with both Na⁺ and Ca⁺ ions.

Using sedimentation procedure, particle size distribution analysis was performed. According to the Romanian standards, the Tulucesti clays, having plasticity index values \( I_p = 33 \text{ to } 42 \) and an activity index \( I_a = 1 \) is sensitive against water. These observations are in good agreement with the
above-presented chemical and mineralogical composition. Using the Voight's correlation between plasticity index versus residual friction angle, the residual internal friction angle values range, \( \tan \phi' = 0.175 \) to 0.230, was obtained. For stability analysis the minimum value corresponding to \( \phi' = 10^\circ \) was chosen.

The laboratory investigations also pointed out the specific macrostructure of the Tulucesti clays. In order to define the peculiarity elements of the studied macrostructure, size distribution analysis of the aggregates obtained by successive cycles of wetting and drying, were carried out. The dispersion effect of these successive cycles can be considered similar to the effect of the natural action of seasonal variation of soil moisture content. A certain quantity of clay was subjected alternatively to drying operations in oven at 60°C followed by the material wetting through aspersion. The soil disintegrated into structural aggregates of variable sizes was subjected to a grain size analysis by sieving.

In Fig. 8, the aggregate size distributions obtained using the above described procedure for 13 drying-wetting cycles are presented comparatively to the domain of the curves for completely dispersed clay obtained using the classical sedimentation. Based on this diagram, the triangular chart presented in Fig. 9 was obtained.

\[ \text{Fig. 8. Macroaggregates size distribution for Tulucesti clay.} \]

To illustrate the specific mechanical behavior of macrostructured Tulucesti clay, triaxial tests have been performed. The triaxial apparatus was the Bishop-Wesley type, developed for controlled stress-path testing. A group of tests were performed in order to point out, for Tulucesti clays, their sensitivity to moisture during shearing (Chirica, 1991, Chirica and Stanculescu, 1994). The samples were initially \( K_0 \) consolidated and sheared afterwards following two different free water supplies:

a) Standard drained tests (CK, DW), with free water contact  
b) Modified standard drained tests (CK, DA), for which the drainage of the samples was opened to the atmosphere without the possibility to drag water during deformation.

\[ \text{Fig. 9. Sequence of crumbling cycles for Tulucesti clay.} \]

For example, on Fig. 10 the volumetric strain variation of a Tulucesti clay sample subjected to shearing in the two mentioned test procedures, are comparatively presented.

One can see that the samples which were in contact with free water source manifested only compression volumetric strains, while the samples which were not in contact with the free water source manifested dilatancy during the shear process, on the same stress-path. In both cases, the volumetric strain values were dependent on the confining pressure and normal stress. This behavior is typical for the structured clays as Tulucesti clay. In the case of this soil category the shearing displacements are leading to relative motions of the structural aggregates still uncrushed, along discontinuity surfaces, together with their opening. These relative movements are bringing about mechanical swelling and, consequently, the increase of the moisture content, by suction. As a result, in those zones where there is a source of free water, the shear strength of the soil is drastically diminished, provoking the failure. This happens within the Tulucesti clay very near from the base of the loessial stratum, in which the free water source is located. In this way, due to dilatancy process, a weak zone, right under the loess, appears. This leads to the detachment of a new plot of soil from Covurlui plateau, triggering, in this way, the detrusive short landslide with the bulging zone between AB6 and B4 boreholes.

**STABILITY ANALYSIS AND REMEDIAL WORKS**

The stability analysis was performed for the profile located at km 6+250 (Fig. 6) where the geotechnical data were available. The considered slip surfaces, regardless of the ground surface configuration, were classified as follows:

1. Short slip surfaces with the breach line on the Covurlui plateau at 70m elevation and with the bulging zone between boreholes AB6 and B4, 175m downstream borehole B6 (denoted T3, T6, T7).
2. Short slip surfaces with breach line between boreholes AB6 and B3 and the bulging zone at the Brates Lake bank (T10, T12, and T14).
3. Long slip surface with the breach line on the Covurlui plateau at 70m elevation and the bulging zone at the Brates Lake bank (T1, T2, T4, and T5).

The stability analysis used the Fellenius slice method in the forward variant as well as in the back analysis. Pairs of the internal friction coefficient, \( \tan \phi' \) and the cohesion, \( c' \) which assure the limit equilibrium at failure were established.
Initially, two ground surfaces, corresponding to the years 1985 and 1993, were considered. In Table 2 the necessary shear strength parameters for different slip surfaces considering both ground surfaces as mentioned above, are presented.

Considering the 1993 ground surface, the back analysis performed for the long slip surface T4 revealed residual shear strength parameter values ($\phi_r'=11^\circ; \gamma_c'=0$) which are very close to those resulted from laboratory tests ($\phi_r'=10^\circ; \gamma_c'=0$). The obtained safety factors (Table 2) confirm the failure mechanism described above.

<table>
<thead>
<tr>
<th>Ground Surface</th>
<th>Slip Surface</th>
<th>$c'$ (kPa)</th>
<th>$\tan\phi'$</th>
<th>FS</th>
<th>$X_{NP}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1985</td>
<td>T1</td>
<td>59.81</td>
<td>0.203</td>
<td>0.861</td>
<td>131.41</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>50.38</td>
<td>0.234</td>
<td>0.749</td>
<td>129.12</td>
</tr>
<tr>
<td></td>
<td>T4</td>
<td>39.97</td>
<td>0.131</td>
<td>1.314</td>
<td>283.12</td>
</tr>
</tbody>
</table>

| 1993           | T4           | 43.84      | 0.192       | 0.909 | 131.81      |
|                | T6           | 29.02      | 0.198       | 0.884 | 131.40      |
|                | T7           | 27.40      | 0.171       | 1.021 | 105.28      |
|                | T10          | 25.44      | 0.164       | 1.067 | 335.59      |
|                | T12          | 15.72      | 0.090       | 1.941 | 251.95      |

| Reshaped profile | T4           | 37.53      | 0.181       | 0.990 |-            |
| Reshaped profile | T6           | 26.66      | 0.267       | 0.656 |-            |

Thus, in 1993 the instability phenomena began with the failure along the short slip surface T6 (FS=0.884), followed by the failure along long slip surface as T4 (FS=0.909). Due to the landslide mechanism previously presented, three main remedial measures were proposed:

1. Excavation in the active zone of the landslide (Fig. 6).
2. A soil improving treatment at the slope toe, bounding the railway, based on modules of Portland cement and gravel Franki piles.
3. Drainage works in order to collect and drain the surface and the percolating water.

In order to properly establish the location of the excavation zones, the neutral point position, $X_{NP}$, for each considered slip surface was calculated (Hutchinson, 1977). The effect of an excavation along the reshaped profile, shown in Fig. 11, was investigated for the slip surface T4.

![Fig. 11. Horizontal thrust diagram along sliding mass for T4 failure surface.](image)

One can see that the safety factor increased for the reshaped profile (FS=0.909 compared with FS=0.909). This increasing is due to the extent of excavations in the active zone of the slip surface only. The passive zone is delimited by the neutral point which is situated about 70m downstream the borehole.

Using the residual strength parameters the diagrams of the horizontal thrusts for T4 slip surface were calculated for the ground surface corresponding to both 1993 post-slide profile and the reshaped one, respectively. The maximum thrust values, situated in the vicinity of sliding mass bulging, 25m downstream of borehole B4, are $E=11500\text{kN/m}$ for 1993 post-slide profile and $E=10300\text{kN/m}$ for the reshaped one, respectively. This means that the excavation decreases with 10.4% the maximum thrust value. As observed in Table 2, the excavation has an unfavorable effect on the short slipsurfaces, with the breach line situated on the plateau, as T6. The stability analysis showed that the excavation couldn't assure alone safety factor greater than unity.

It is to be mentioned that the latest inclinometric logs, after the excavation works (September 1994), indicate displacements only for the base of the slope. Thus, it strikes out that the stability state of the slope has to be improved along the short slipsurfaces, situated at the toe of the sliding mass, such as T10.

INCLINOMETRIC MEASUREMENTS PERFORMED FOR THE SURVEY OF THE MOVEMENT EVOLUTION

The Railways Study and Design Institute of Bucharest coordinated the engineering studies. After the completion of these studies and the rehabilitation, the circulation has been re-opened under the speed restrictions and continuous inclinometric deformations survey in order to avoid the accidents. The Research and Engineering Institute for Environment of Bucharest contribution was focused mainly on the inclinometric survey of deformations.

In order to determine the slide planes and the movement evolution 23 boreholes were equipped with aluminum inclinometric tubes. The survey was carried out on profiles, each of them comprising 2 to 6 boreholes. The alignments of profiles are perpendicular to the railway following the general landslide direction. The A, B, and C profiles have 5 to 6 boreholes, two of them being on each side of the railway, 3 on the unstable zone of landslide and one on high stable terrace. The D, E, and F profiles have only 2 boreholes on each side of the railway. The surveyed zone is between km 6+050 (Profile A) and km 6+550 (Profile F) at railway.

The deformation measurements were performed with SINCO biaxial inclinometric device equipped with SINCO servo-accelerometers. The inclinometric equipment has a 0.1mm/m sensibility and can assure a 0.2 to 0.3 mm/m precision. For 30m deep boreholes a maximum error of ±6mm at the ground surface is accepted.

These inclinometric measurements allowed the slide plane depth and the movement direction and evolution to be determined.

The inclinometric measurements emphasized the existence of one main slide surface found at rather great depths. This main movement does not exclude some others observed within the
slope, above the main surface. At the railway, the main slide plane was found at 10 to 15m depth while on the slope the depth is greater than that. The greatest of them was found at the C3 borehole and is about 30m. In Fig. 12 the diagrams recorded at this location are presented.

The slide plane is very well marked at 30m depth. The displacements are uniformly amplified with the time. Together with the inclinometric measurements, the water level was recorded. The high piezometric levels in the C4 and C5 boreholes are to be noticed. Both of them are artesian. A lot of observations allowed us to state that the high levels correspond to the pore-water pressures at the main slide surface. The water level measurements performed during 1994 did not indicate significant variations of the water pressure, the observed differences being less than 3m.

The specificity of the studies carried out for this landslide, while the railway traffic was maintained, determined a high inclinometric measurement frequency in order to allow the prediction of movements evolution. The time evolution of the displacements recorded at the profile C is presented in Fig. 13.

The movements presented were observed during December 1993 through February 1995; afterwards the slope was rather stable. The total displacement was 70mm.

The consolidation works began at the end of 1993 and continued during the first half of 1994, then was stopped. They consisted mainly of upper part excavations and lower part embankments between km 6+050 and km 6+250 at railway. Drainage works and soil improving treatment at the slope toe were designed but they have not been completed until now. In order to evaluate the efficiency of the consolidation works, 5 inclinometric boreholes (BA1 to BA5) were placed at km 6+150 in the middle of the trial zone.

The measurements performed at borehole BA4 in this profile (Fig. 14) did not show any displacement during the second half of 1994. The diagrams were in the normal error range. All other boreholes on this profile presented the same condition.

REFERENCES


