

Landslide25_05**Studying the slope state at 4 M.Hrinchenka Street in Kyiv and technical solutions developing for its stabilization**

V. Tytarenko (*The State Enterprise “The State Research Institute of Building Constructions”*), **O. Tytarenko** (*National Technical University of Ukraine*), **V. Polevetskiy** (*Yuriy Fedkovych Chernivtsi National University*), **I. Zhemelinsky** (*National University of Life and Environmental Sciences of Ukraine*), **Y. Berchun** (*Institute of Telecommunication and Global Information Space NASU*), **I. Kaliukh** (*Institute of Telecommunication and Global Information Space NASU*)

SUMMARY

This paper presents a comprehensive study of a soil failure at 4 M. Hrinchenko Street, Kyiv, and the development of engineering measures for stabilizing landslide deformations that threaten the surrounding multi-story buildings. The research methods included visual inspection of active gully erosion sites and damage to a parking facility near a residential high-rise; testing of the physical and mechanical properties of slope soils for refining the initial boundary conditions and subsequent mathematical modeling of the stress–strain state of the soil mass. For these purposes, the Plaxis software package and a nonlinear soil model governed by the Mohr–Coulomb strength criterion were employed. Visual inspection revealed that the average slope angle is 20–22°, with a width of about 34 m and a length ranging from 13 to 52 m. Three active gully erosion centers were identified, along with the presence of “loess karst.” Its development is driven by the steep relief of the slope and the influence of groundwater and surface runoff. Numerical modeling showed that the overall slope stability is not ensured. At present, the existing landslide does not yet compromise the stability and reliability of the surrounding high-rise development. However, failure to implement emergency mitigation and stabilization measures could significantly worsen the condition of the territory adjacent to the building, causing damage to engineering networks and utilities (for example, a medium-pressure gas pipeline located at the foot of the slope).

Introduction

The study object is a portion of a mainly landslide and partially landslide-hazardous soil slope, the steepness of which is 20-22° on average. The slope height is 35-37 m. In 2008, high-rise buildings were erected at the slope upper part. During engineering and geological surveys on the slope portion and adjacent territory carried out in connection with the structures construction, the development of dangerous geological processes and phenomena is noted. On the slope a landslide occurred. The landslide had a width about 34 m and length of 13÷52 m (Fig. 1).



Figure 1 Landslide state in the slope upper part.

The distance from the failure edge to the 3-4-storey parking lot of high-rise buildings was about 18.5 m. Comparing the landslide failure edge state in late May and early July, the appearance of additional soil surface deformations in a form of surface cracks (marked by wells and parking space No. 42) formed at a distance of 3-5 m deep into the territory (on the plateau) is noted. In geomorphological terms, the surveyed area is on a loess remnant called "Batyieva Hora" on the right slope of the Lybid River valley with absolute elevations of 119.5 - 155.5 m. The area is situated on the former cotton-spinning factory No. 22 territory between Hrinchenka St. (the slope lower part) and Amosova St. (the slope upper part) and borders at its southern border with the Baikove Cemetery plot 42 and at its northern border with the Rosa JSC factory territory. Here a process of systematic planar washout, i.e., the soil layers erosion by watercourses, is developing. On the surveyed slope an active gully formation is observed. According to the results of the surveys, three foci of active gully formation are identified. On the studied territory a "loess karst" is found. It is associated with the presence of easily eroded varieties of loess-like sandy loams lying from the surface near the slope surface. Its development is also facilitated by the rather steep relief of the soil slope and active atmospheric and surface waters. It manifests itself in the form of the formation of underground funnels, well-like dips, caves and, as a result, failures such as shallow landslides, small blocks slumping and loose sediments sliding. The slope toe is secured by a retaining structure made of 1.5-2.5 m high bored piles with a diameter of 600 mm and lined with rubble stone. The asbestos-cement trays are laid along the slope to drain surface water. At the time of surveying, almost all trays were clogged with debris and soil, and many of them were broken, which led to the increased erosion processes on the slope. In the slope lower part, along the existing retaining wall, the multiple groundwater wedges into the natural surface are observed in the form of springs, gullies and streams with a flow rate of 0.01 to 0.1 l/sec. There are the active suffusion removals of sandy material (mechanical suffusion) from the existing sources, soil porosity increasing and density decreasing, which, along with the resulting surface runoff, lead to the significant soil falls and loose sediments sliding in the central and northern site parts. The issues of prevention and counteraction to geohazard accidents were considered in many works of such domestic and foreign researchers as (Hadiatska, 2019, 2020, 2021), (Ivanik, 2023, 2025), (Kaliukh, 2023, 2024, 2025), etc.

Numerical modelling of the soil massif stress-strain state

The most dangerous engineering-geological cross-section according to calculations (the stability coefficient for the 1-1 sliding surface taking into account the forces along the sliding surface before the area

development is $k_{st}=0.843$) before and after the development is considered. To assess the stress-strain state (SSS) of the soil massif in accordance with DBN V.2.1-10-2009 Chapter 8, the Plaxis software package is used. To model the soils SSS, a nonlinear model, the strength criterium of which is described by the Mohr–Coulomb law, is used. The soil is considered as an elasto-ideally plastic material, the deformation of which accords with the Prandtl diagram. This means that the yield surface does not change with the yield strain increasing, i.e., there is no strengthening. For the Mohr-Coulomb model correct using, the following values of its main parameters are required: 1) drained, undrained or non-porous soil behaviour; 2) specific gravity γ_{sat} in the natural state and saturated unit weight γ_{sat} (are specified taking into account the possible rise of the groundwater level); 3) coefficients of permeability k_x and k_y , which have the physical significance only for undrained soil behavior; 4) deformation modulus E ; 5) Poisson's ratio ν ; 6) soil internal friction angle φ ; 7) unit cohesion c (in this case, it is the structural cohesion c); 8) dilatancy angle ψ . The strength parameters (c and φ) of the model for soils are assigned based on the data of the engineering geological survey report. The stiffness parameters (E) for soils are also assigned according to the data of the engineering geological survey report. The Poisson's ratio ν is taken for loams 0.35 – 0.37 in accordance with DSTU B V.2.1-4-96. In this case, the smaller values of ν are taken at the higher soil density. The effect of hydrostatic water pressure is taken into account when setting the corresponding groundwater level. To model the reinforced concrete pile shaft and grillage a linear elastic model is applied. This model corresponds to Hooke's law of isotropic linearity of elasticity. The model includes two elastic parameters of stiffness, namely the modulus E and Poisson's ratio ν . These values are determined in accordance with DSTU B V.2.6-156:2010. For spatial clusters of this type, the non-porous option is used. In this case, the initial and additional pore pressures in clusters of the corresponding type will not be taken into account. To ensure the necessary confidence level of the calculation results, when using the average characteristics, it is sufficient to use the coefficients of reliability as to soil only for the parameters, c and E . The slope calculation is carried out according to the scheme, which involves changing the soil strength characteristics until the slope reaches a state of limit equilibrium. The load from the above-foundation part of the building is set evenly distributed along the foundation edge. The building foundation is assumed to be absolutely stiff (the sliding processes impact is assessed for the building as a whole, therefore the building stiffness is not taken into account). The boundary conditions correspond to the natural operation conditions of foundations and piles under load, namely: all nodes located on the lower plane of the calculation zone shall be fixed immobile; nodes at the side surfaces cannot have any horizontal displacements. The design finite element scheme is shown in Fig. 2. It consists of 800 elements, 6613 finite element nodes and 9600 stress points; the finite element average size is 3.78 m. Such elements quantity and dimensions satisfy the calculation result: the further reduction does not lead to the calculation accuracy increase.

Results

The calculations reveal that the stability of the building on pile foundations is ensured. This statement follows from the fact that all possible slip lines are within the soil massif located lower down the slope (Fig. 3). The numerical modeling of the stress-strain state of both the slope soil massif and the building base allows to draw the following conclusions: the slope overall stability is not ensured and any impact can lead to the landslide phenomena development; the bulk, deluvial and loess soils and lying lower the building on the slope are landslide-prone; to stabilize the landslide phenomena, it is necessary to implement a number of engineering measures listed in the conclusions to this report; the slope SSS analysis and its stability assessment after a landslide and after the repeated landslide phenomena possible in the future show that the stability of the building on pile foundations is ensured. This follows from the fact that all possible slip lines are within the soil massif lying lower on the slope.

Conclusions and recommendations

1. The slope area is a zone of an active gully formation. The increasing landslide processes emerge with the separation of blocks in diluvial and loess rocks and subsequent rockfalls and block slumps. "Loess karst" is observed. In the lower part of the slope, suffusion of a mechanical nature occurs by sandy material piping

from existing sources. The active morphogenesis of the studied area takes place because of violating the rules of drainage structures and storm drains operation.

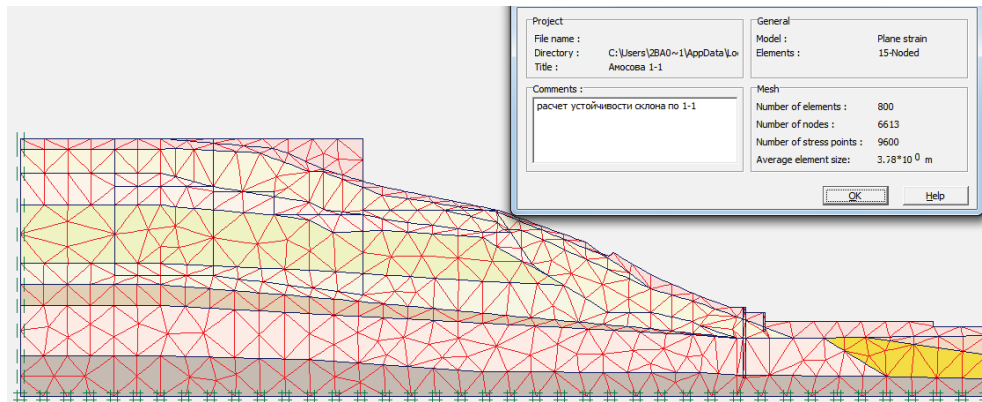


Figure 2 Design finite element scheme for the slope. Key: slope stability calculation for 1-1.

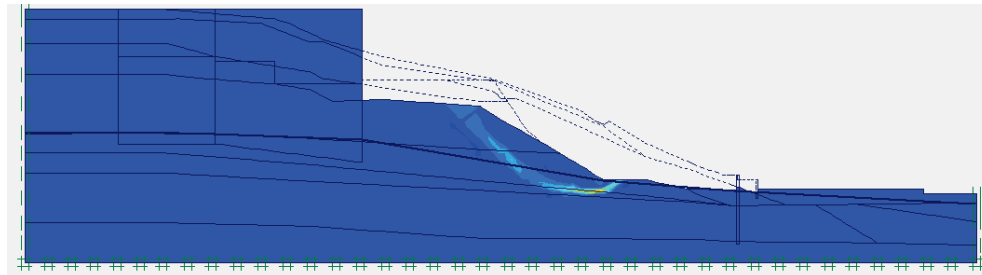


Figure 3 Design finite element scheme for the slope. Key: slope stability calculation for 1-1.

2. Based on the performed calculations on the determining the overall stability of the slope along with a high-rise building, it is generally established that the slope stability is not ensured, but the stability of the building on the pile foundations is ensured. This follows from the fact that all possible slip lines are within the soil massif located lower down the slope. Thus, the existing landslide and its further development do not affect the stability and reliability of the high-rise building on Amosova Street. On the other hand, the non-fulfilment of measures on eliminating the slope emergency state and its stabilizing can significantly worsen the condition of the territory adjacent to the building and cause damage to engineering networks and communications, which can also lead to the engineering networks operation failure in a high-rise building.

3. The priority measures on eliminating the slope emergency state (landslide) include intercepting rainwaters from the destroyed stormwater drainage system. First, it is necessary to install an emergency water disposal system. The following works should be included: inspecting the integrity of pipes that have an outlet into the landslide failure wall; inspecting the stormwater drainage system at the slope foot; laying pipes or industrial trays to the slope foot for connecting the stormwater drainage system at the slope foot and the pipes that exit into the landslide failure wall.

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