Geotechnical Aspects of Design and Construction of the Mountain Cluster Olympic Facilities in Sochi

Les aspects géotechniques des projets et de la construction des sites olympiques situés dans les pays montagneux autour de la ville Sotchi

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ABSTRACT: Construction of Olympic Facilities in Krasnaya Polyana, mountainous area near Sochi, was a real challenge to Russian geotechnical engineers for the lack of construction expertise in the area as compared to the Alpine mountain resorts with extensive record of such activities. The geotechnical challenges of construction in this mountainous cluster include, as follows:
- assessment of deformation and strength parameters of eluvial and half-rock soils, including rubbly and gravely ones;
- selection of reliable techniques to analyze stability of natural, transformed and strengthened slopes;
- design of footings and counter-slide structures in order to ensure adequate safety accounting seismic actions and soil condition variations.
The paper describes three case histories of Olympic construction projects within the mountainous cluster to illustrate how the above problems were solved.

RÉSUMÉ : Les travaux de construction des bâtiments olympiques dans le massif montagneux de Krasnaya Polyana aux environs de Sotchi ont causé nombre de difficultés aux géotechniciens russes. Cela s'explique par le manque de l'expérience nécessaire en ce domaine dans cette région, contrairement au cas des stations de ski des Alpes, qui ont une plus ancienne et plus riche histoire de développement. Les problèmes géotechniques essentiels liés à la construction dans la dite région consistent en les points suivants:
- évaluation des paramètres de déformabilité et de résistance des sols éluviaux et en partie rocheux, incluant des remblais rapportes ;
- choix de méthodes pertinentes de calcul de stabilité des pentes naturelles, aménagées et renforcées;
- conception de fondations et de structures installations para-glissements garantissant un niveau adéquat de stabilité, y compris pendant les séismes ou un changement d'état des sols.
Dans cet article, on décrit trois études de cas relatifs à la construction des installations olympiques dans la région montagnarde pour illustrer comment ces problèmes ont été résolus.

KEYWORDS: slope stability analyses, counter-slide structures.

1 INTRODUCTION

Erection of Olympic facilities within the Sochi mountainous cluster (Krasnaya Polyana area) forced construction engineers, particularly geotechnical engineers, to face quite a few complicated problems due to tight deadlines, absence of expertise of such large-scale projects in the area, complicated geological conditions and seismicity. The situation dictated that the leading research organizations such as Gersevanov Research Institute of Foundations and Underground Structures would take part in the design of some facilities and structures along with expert evaluation of the project designs. The paper describes the main challenges of the above work on three Olympic projects and their respective solutions.

2. GEOLOGICAL ENVIROMENT

The project (highway from Krasnaya Polyana to "Roza-Khutor" ski resort", bobsleigh/luge track and a complex of ski-jumps) are allocated on the left-bank slope of Mzymta river and plateau expansion of Aigba ridge offspur at 950…1100 m altitudes with the valley bottom at 485…490 m. The terrain varies from gentle 5…15° slopes to steep 35…40° slopes. The area is cut with streams – tributes to Mzymta.

Geologically there are features from quaternary to underlying Jurassic deposits. The latter are mainly represented by argillites, interlayered with sandstones and limestones. Stratified partly outcropping argillites occur below 10…20 m depths (Fig. 1).

In terms of geotechnical engineering the main rocks are quaternary, mostly formed from of argillite bedrocks. Closer to the surface gravelly clay-filled soils contain more clay and less rock debris and gravel (Fig. 2).

Fig. 1. Outcropping argillite

Such grain size composition complicates any tests to determine soil deformation and, especially, strength parameters. Therefore, pillar shear test was the main method along with large-size sample (∅40 cm) direct shear test that required application of large-size test equipment. Additionally, pillar tests show the natural anisotropy of argillite properties due to its stratified structure.

Fig. 2. Gravely clay soil

At the initial stage gravely soil strength parameters were determined indirectly from results of crushing dry soil in a pebble mill (DalNIIS technique, 1989). Then the obtained parameters were corrected, because the natural slope stability analyses yielded faulty results (slope stability factor $k_{st} < 1$). Later the obtained experience made it possible to correlate the values of soil strength parameters with soil composition and state and thus to identify essential errors.

Essential reduction of soil shear resistance after moistening was a specific feature of the terrain. The relief usually prevented long-term moistening by torrents and/or melting snow. However, soil strength parameters variation had to be taken into account in stability analysis as an action.

Seismicity is yet another special action. Design seismicity of the terrain is reportedly from 8 to 9 points. In the analyses the seismic acceleration was generally assumed to be horizontal, but in some cases a more unfavorable direction had to be taken into account i.e., at 30° versus the horizon.

3. KRASNAYA POLYANA – ROZA-KHUTOR ROAD

Research and technological support of the motor road project was the authors' first effort. Therefore, the basic analytical and design concepts were tested on this very project.

The first soil data was obtained by DalNIIS method (1989) and initially looked dubious. It was especially so for gravel and pebble soils with clay fill, for which internal friction angle was 23.8° and 26.6°, cohesion 13.6 kPa and 11.3 kPa respectively. For coarse-grain soils the values of φ were evidently underestimated. This fact was proved by natural slope stability analysis. For some road cross sections the value of k_{st} with characteristic values of c and φ dropped down below 1.0 and even below 0.8.

This was demonstrated after two stability analyses: by the method of variable level of shear-strength mobilization (MVLM), proposed by the authors (Fedorovsky, Kurillo, 1998, 2001), and by finite elements analysis (Brinkgreve, Vermeer, 1998). Both methods MVLM and FEM (PLAXIS) yielded close k_{st} values that were much lower than those obtained with the well-known Bishop, Morgenstern-Price and Janbu methods, applied by the surveyors. This fact demonstrates that the new methods are certainly better for the analysis of essentially heterogeneous soils. The new methods are also more accurate, as is shown by comparing solutions of problems, having exact solutions (such as bearing capacity problems).

After φ and c of the two above-mentioned soils were corrected to 36°, 32.6° and 16.4 kPa, 19.7 kPa respectively, all the analyzed sections yielded stability factor slightly over 1. This was due to the fact that in the least stable sections the critical slip-lines passed through these very two engineering geological elements..

The road is constructed by cutting into the rock massif and sometimes by filling soil and by enlargement toward the lower slope. The objective was to select such strengthening solution for the upper and the lower slopes that would ensure the minimum value of k_{st}, at least 1.15,for the main load (including 10 kPa load on the road proper) and 1.05 for the special (seismic) load. Where necessary, the upper slope was to be retained by anchors, inserted into the primary rock (argillite). In order to minimize the impact on the natural environment the upper slope retaining wall was made rather steep (60° versus the horizon) and 8…16 m high. In order to ensure adequate stiffness and strength of the slope plane 6..8 m long soil nails were proposed (Fig. 3).

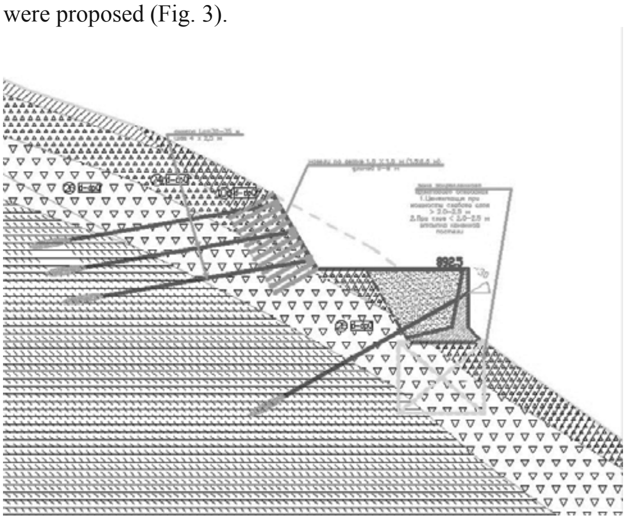

Fig. 3 Counter-landslide structures for the motor road.

Where the lower slope fill is insignificant, no extra measures were required. However, at some locations an angle-shaped retaining wall, strengthened by a row of anchors, was to be erected. However, it was insufficient for one of the cross-sections, as the wall was supported by soft soil. It was proposed to replace the soil by broken stone fill or to strengthen it by grouting (рис. 3).

Notably, application of anchors "neutralizes" both local landslides of the upper and lower slopes along with the global ones, initiated above the road and ending below it.

The anchors were directly simulated in PLAXIS (with the account of transfer from 3D to 2D). In MVLM method the plane, to which the anchors are fixed, is loaded to simulate stressed anchor action on the slope. Both methods demonstrated that the pulling force, applied to 2.5 m spaced anchors (along the road), per 4 m along the slope is 40…45 tons.

Just a few words on seismic numeric simulation. Russian standards recommend to apply proportional inertia forces to soil weight with AK_1 factor. Here A depends on the terrain seismic intensity(A = 0.2 for magnitude 8), and K_1 depends on allowable soil deformations. If a soil slope is viewed as a structure with limited (landslide) deformations then K_1 = 1. If then the road subgrade is considered separately from the counter-slide structures and the plastic non-destructive deformations of the structure are allowed then K_1 = 0.25. Finally, it is only logical to assume (geometrical) mean value of $K_1 = \sqrt{1\cdot 0.25} = 0.5$, even more so that the result coincides with recommendations of Eurocode 8.

4. BOB-SLEIGH/LUGETRACK

This project was more difficult than theprevious one due to considerable relief altitude drops and to diverse hydrogeological conditions.

Bobsleigh/luge track (BLT) is located in the lower part of the Aibga ridge northern slope near Krasnaya Polyana.

Orographically the BLT terrain is located in a mid-mountain relief zone with 650-800 maltitude drop. Within the area next to the survey terrain Esto-Sadok and East-Achikhinsky fault zones occur. The South-Esto-Sadok fault passes at the south of the surveyed zone close to the “Bean Storage Area”. One of the feathering faults, occurring from north-west to south-east, passes across the northern end of the designed trough. The massif is water-logged via aquifer zones all the way down to the investigated depth, the water heads correlate with the cut depths through the surface valley due surface flows of the Shumikhinsky stream.

Geological slope cuts are mainly represented by high density gravely clay loam or by gravely soils with clay loam fill. The clay loams and clay loam fills feature liquid-plastic to hard consistency.

At 16-40 m depths the quaternary deposits are underlain by low-strength argillites. Depending on the water table the argillites and their fills feature liquid plastic to hard consistency.

The seismicity of the construction project location is 8.5 points as per micro-seismic zoning.

The survey identified three slope terrains, on which development of landslide processes is possible under design seismic action. In order to confirm slope instability the authors performed verification analysis of the above-mentioned slope terrains with the help of PLAXIS as they were and in the case of BLT structures erection. According to the analytical results the stability factor was below the admissible level of 1.1 for 8.5 points seismic action(Figs. 4, 5).

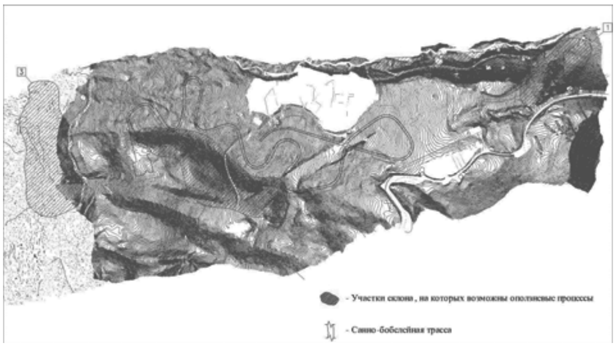

Fig. 4. Topographic map of BLT terrain. Black domains - landslide prone zones; the curved line - bobsleigh/luge track

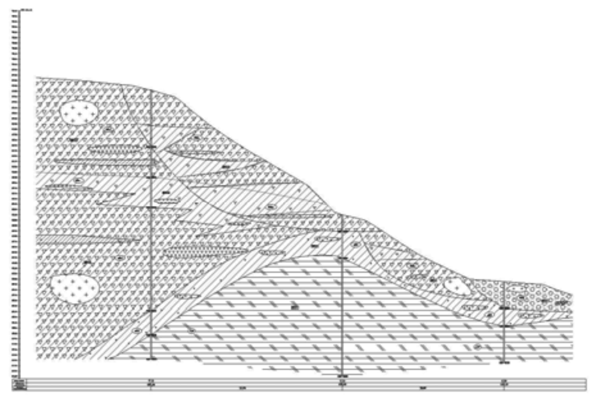

Fig. 5. Land-slide prone slope cross section at terrain 1

Then the authors proposed measures to provide for the required values of stability factor i.e., to erect retaining walls of various configurations, depending on the internal forces in them (one or two rows of bored piles, groups of bored tangent piles with the stiff pile capping beam).

Herein, application of piles as counter-slide structures shall be discussed. Spaced piles, located as a row across a slope would not let soil move between them at whatever landslide pressure (the effect of “non-pushing through”, Fedorovsky, 2006) with the critical clearing between piles being larger the greater is the internal friction angle. However, the drawback of such pile strengthening consists in that the bending forces in the piles are so high that often surpass their bending strength. Therefore, in difficult cases the bending moments are reduced by respective measures (pile heads anchoring), or the piles stiffness or strength are increased (larger diameter up to 1.5 m) or installation of buttresses or several piles instead of single piles. In this case there were proposed bored tangent piles with a strong pilework on top. Bored secant piles with dedicated reinforcement are more effective.

Pile walls along the slope feature one more advantage. If their spacing across the slope is less than the wall length then the active (landslide) soil pressure is, as a rule, less than that of the ultimate thrust (Nazarova et al., 1995). The above structures were widely applied for the next facility, discussed below.

5. SKI-JUMP COMPLEX

The complex of K-125 and K-95 ski-jumps geological environment is similar to that of BLT, however, the altitude drops are greater, but hydro-geological situation is better.As different from BLT the counter-slide structures are partly combined with footings of the proper ski-jumps, of the landing slope, of the start and the referee towers.

Fig. 6. Ski-jump site at the beginning of construction operations

Fig. 7. The same terrain during footing erection

This is due to some factors. Firstly, the initial slope (Fig. 6) has k_{st} = 1.04 for soil design parameters while k_{st}< 1.0 for seismic conditions. The ski-jump track is located in a cut 8…10 m deep that undercuts the side slopes and deteriorates the landslide situation (Fig. 7).

In order to overcome these difficulties buttress rows of 3…5 of 0.88 m dia bored secant piles were selected, with some of these piles being anchored at their heads (Fig. 8). The pile fabrication process is shown on Fig.7.

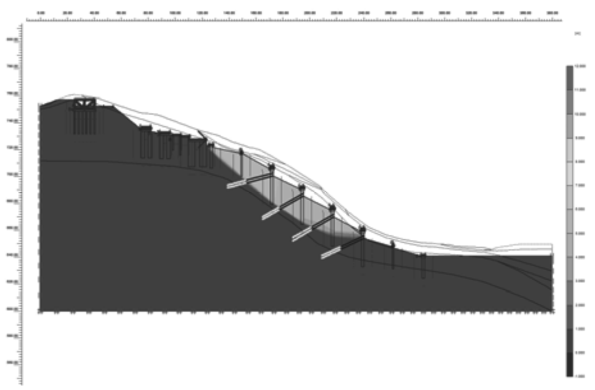

Fig. 8. Analytical scheme of a group of ski-jump footings in PLAXIS

The analysis of the row of buttresses was made in two steps. At the first step it is proved that the limit resistance of the structures to soil flow around is greater than the active (landslide) pressure. To this end a FEM analysis was made of the push-through pressure with the layer depth, represented by the value of pressure on rear side of the row. At the second stage FEM analysis was made either, this time reduced soil strength parameters technique was assumed to assess the slope stability factor with piles and anchors present.

Beside the ski-jumps proper, engineering protection of the terrain was to be taken care of. In order to reinforce the side slopes, retaining walls on piles were proposed. For lower slopes soil nails were assumed in the central portion rather than along the whole height. Such reinforcement divides the slope in two short segments: the upper and the lower one, with the stability factor for each one being greater than that of a single deep landslide of the whole slope.

6. CONCLUSIONS

Application of up-to-date slope stability analysis methods enabled improvement of Olympic facilities project designs in the Sochi mountain cluster in terms of engineering protection of the terrain and of the facilities.

For landslide control structures, sometimes combined with footings, various options were proposed, adjusted to local conditions: soil nails, anchors, retaining walls on subsoil or on piles, rows of piles and buttresses. These structures were applied as combinations rather than separately.

The above analytical techniques, FEM particularly, proved to be effective in the analysis of interaction of landslide-control structures and footings with soil.

Combinations of all these factors ensured construction of Olympic projects to meet the tight deadlines and to provide their adequate safety.

7. REFERENCES

Brinkgreve R.B.J., Vermeer P.A. (eds) 1998. *PLAXIS.Finite Element Code for Soil and Rock Analyses*. Balkema

DalNIIS. 1989. *The method of strength and compressibility assessment for macrofragmental soils with silty and clayey fill and silty and clayey soils with macrofragmental inclusions*.(in Russian). Stroyizdat, Moscow

Fedorovskii V.G. and Kurillo S.V.. 1998. Method of a variable level of shear-strength mobilization for calculation of the strength of soil masses. *Soil Mechanics and Foundation Engineering* , 35 (4-5), 121-126

Fedorovsky V.G. and Kurillo S.V. 2001. Method of a variable level of shear strength mobilization for calculation of the strength of soil masses. *Proc. XVth ICSMGE*, v.1, Stambul, 687-690

Fedorovskii V.G. 2006. Limiting pressure on rows of strip plates and the "no-force-through" effect. *Soil Mechanics and Foundation Engineering* 43 (3), 85-91

Nazarova N.V., Stavnitser L.R., Shvets V.B. 1995. Characteristics of longitudinal retaining walls used as slide-restricting structures *Soil Mechanics and Foundation Engineering* 32 (5), 167-173