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Review paper
Doi 10.61892/STP202601042C
ISSN 2566-4484

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CONSTRUCTION OF THE IBARAC TUNNEL AS PART OF THE ROŽAJE BYPASS PROJECT IN MONTENEGRO

Abstract:

The Ibarac tunnel was excavated as part of the Rožaje bypass project in Montenegro. Nearly half of the excavation was carried out in glaciofluvial sediments consisting of brown sandy clay with gravel and rock fragments. As the original design did not anticipate tunneling through such deposits, the subsequent adoption of overly optimistic geotechnical parameters led to several collapses during the construction phase. A major collapse occurred within a section containing a parking niche at km 0+473. Although the overburden at this location exceeded 100 m, deformations reached the surface, forming a vertical channel with a diameter of 26 m. This paper proposes a rehabilitation solution using a pre-grouting technique.

Keywords: Ibarac tunnel, glaciofluvial sediments, collapse, rehabilitation, pre-grouting.

ИЗГРАДЊА ТУНЕЛА ИБАРАЦ КАО ДЈЕЛА ОБИЛАЗНИЦЕ ОКО РОЖАЈА У ЦРНОЈ ГОРИ

Сажетак:

Тунел Ибарац ископан је као дио пројекта обилазнице око Рожаја у Црној Гори. Скоро половина ископа изведена је у глациофлувијалним седиментима, који се састоје од смеђе пјесковите глине са шљунком и фрагментима стјене. Како оригинални пројекат није предвиђао ову средину, накнадно су за њу усвојени превише оптимистични параметри што је узроковало неколико урушавања током фазе изградње. Главно урушавање догодило се унутар деонице која садржи паркинг нишу на км 0+473. Иако је надслој на овој локацији већи од 100 m, деформације су досегле површину, формирајући вертикални канал пречника 26 m. Овај рад предлаже решење за санацију коришћењем технике претходног ињектирања.

Кључне ријечи:Тунел Ибарац, глациофлувијални седименти, колапс,претходно ињектирање.

1. INTRODUCTION

The Ibarac tunnel (1,193 m) and the Carine tunnel (692 m) were excavated as part of the Rožaje bypass in Montenegro. The design specified that evacuation tunnels, following the same gradient, be constructed at a distance of 20 m from the main tunnel tubes. Nearly half of the excavation for both tunnels was performed in glaciofluvial sediments consisting of brown sandy clay with gravel and rock fragments. During excavation, several collapses occurred in these materials furthermore, ground pressure caused excessive deformation of the installed primary support, requiring the reprofiling of several tunnel sections.

Despite these significant problems, the contractor successfully completed the primary and secondary linings in the Carine tunnel. However, construction of the Ibarac tunnel was halted due to a major collapse during excavation in glaciofluvial sediments. Although the overburden at the collapse location exceeded 100 m, deformations reached the surface, forming a vertical channel and a crater 26 m in diameter. Construction was suspended primarily due to the high costs of rehabilitating this collapse and the need to reprofile multiple sections of the main and evacuation tunnels.

The problems encountered during construction can be attributed to the leaching of fine soil particles caused by the intensive flow of groundwater toward the tunnel excavation. This loss of fine particles destabilizes the soil skeleton and causes a rapid increase in pore water pressure, leading to the deformation of the existing primary support and, ultimately, to excavation collapse.

This paper proposes a rehabilitation plan for the Ibarac tunnel collapse using a pre-grouting technique. The implementation of this technique, which strengthens the surrounding soil and is often employed during excavation in water-saturated soils, is described in detail.

2. EXCAVATION OF THE IBARAC TUNNEL

Excavation of the Ibarac tunnel from the entrance portal advanced through glaciofluvial sediments from km 0+317 to 0+507, from km 0+552 to 0+574, and from km 0+592 to 0+631, where progress was halted due to increased water inflow, high convergence, buckling of anchor plates, and cracks in the primary tunnel support. The contractor decided to move the excavation to the exit portal; however, shortly after equipment and workers were relocated, the primary support in the parking niche area (from km 0+454 to 0+506) collapsed.

Excavation from the exit portal continued through glaciofluvial sediments from km 1+277 to 1+162, from km 0+811 to 0+803, and from km 0+682 to 0+631. The contractor advanced to km 0+675, where increased groundwater inflow was recorded at the excavation face. Shortly after, at km 0+651, an outflow of more than 50 m^3 of silty material occurred from the crown area. The inflow of water from the top heading causes the leaching of fine soil particles, with an estimated leakage rate of 20 l/s.

Based on the geological survey reports, new geotechnical models were developed, and a revision of the Ibarac tunnel project was submitted with the following parameters for glaciofluvial sediments: $\gamma = 25.0$ kN/m³; $\varphi = 21.25°$; $c = 79$ kN/m²; $Ev = 91$ MPa. In the project, calculations were performed using the PLAXIS 2D software package. The Mohr-Coulomb failure criterion was adopted, and the relaxation factor λ was determined based on the overburden height, material strength, tunnel geometry, and excavation step.

Excavation of this section was performed using two 15 m long pipe roofs, each consisting of 29 pipes (114 mm diameter) with a 5 m overlap. The support system consisted of 20cm thick shotcrete reinforced with two layers of Q 335 wire mesh and lattice girders 95/22/32 spaced at 1.0 m intervals. Systematic anchoring was implemented using 6.0 m long IBO bolts in the crown and 9.0 m long IBO bolts in the bench, both on a 1.50 x 1.0 m grid. Following the implementation of these measures, excavation advanced to km 0+651, at which point significant deformations were observed in the primary support between km 0+651 and 0+671, which manifested as longitudinal and transverse cracks across the entire profile. Due to compromised stability, additional stabilization measures were taken, including closing the temporary foundation arch, and the systematic installation of IBO self-drilling anchors (Ø32 mm, 9-15 m long) and IBO anchors (Ø51 mm, 9m long) in the area of larger cracks. After the stabilization of the primary support, the contractor proceeded to the breakthrough area from km 0+651 to 0+631. In this area the top heading was excavated in phases, secured by the specified primary support with two 15 meter long pipe roofs and included the closure of the temporary footing arch.

Upon completion of the tunnel breakthrough, the contractor began work on the section previously excavated from the entrance portal, where a collapse had occurred within the parking niche area (from km 0+454 to 0+506). After cleaning debris and mud from the tunnel profile, geodetic monitoring of the primary support system was conducted. This monitoring revealed displacements exceeding 40 cm between km 0+619 and 0+608. Given that these values significantly surpassed the design limit of 15 cm, the existing support had to be reinstalled. The support system was reinstalled in one-meter intervals. Once the old lining was removed and 3 meters of new primary support were installed, work proceeded with the construction of the temporary footing arch.

Rehabilitation of the primary support progressed until km 0+473, when the collapse of the double pipe roofs occurred. This structural failure made further top-heading excavation impossible, and progress was halted. Soon there was a large influx of underground water and then the tunnel collapsed. The material filled 40 m of the tunnel tube, forming a vertical channel 100 m high to the surface, where a crater with a diameter of 26 m was formed, Figure 1.

Figure 1. Crater on the surface of the terrain as a result of the collapse in tunnel (photography by author).

The contractor implemented additional measures to secure the tunnel excavation and developed a rehabilitate proposal with geotechnical parameters for the disturbed glaciofluvial sediments: $\gamma = 20.0$ kN/m³; $\varphi = 16°$; c = 7 kN/m²; Ev = 40 MN/m². These soil parameters are significantly lower than those originally specified in the project documentation. The contractor attributes this discrepancy to the soil displacement following the collapse.

Reservations were raised regarding the proposed rehabilitation strategy, as a fully formation of a load bearing ring around the tunnel could no longer be guaranteed following such a large collapse. Despite this, during initial rehabilitation efforts, the contractor successfully established a connection through the collapsed zone using a reduced excavation profile. The pilot tunnel ensured free groundwater flow and reduced ground pressure. As shown in Figure 2, the reduced profile was stabilized with lattice girders, reinforcing mesh, shotcrete, and anchors. However, works were soon suspended due to a lack of funding.

Figure 2. Reduced profile, through the collapsed material of the Ibarac tunnel (photography by author).

3. DESCRIPTION OF STABILITY PROBLEMS IN GLACIOFLUVIJAL SEDIMENTS

While primary support was installed along nearly the entire length of the Ibarac tunnel, excavation was hampered by several major collapses. Investigating the root cause of the deformations following support installation is critical. Although the glaciofluvial sediments initially appeared stable, Figure 3, it is evident that their geotechnical parameters were overestimated and that a temporary footing arch should have been systematically implemented. Nevertheless, the weakened tunnel subgrade remained stable until the ingress of groundwater. This groundwater influx not only exerted additional hydraulic pressure but also triggered the leaching of fine particles, ultimately compromising the soil skeleton and leading to instability.

Figure 3. Tunnel face in glaciofluvial sediments, Ibarac tunnel (photography by author).

In the case of fine-grained soil, the volume and intensity of groundwater inflow during excavation are decisive factors determining the stability of the tunnel excavation. Immediately after excavation, a hydraulic gradient is established between the aquifer levels as yet unaffected by the works and the tunnel face, where fluid pressure is atmospheric (zero). This gradient generates a high velocity flow capable of entraining particles, which means that the tunneling process itself increases soil permeability through leaching.

In the initial state, the hydrostatic pressure at the tunnel level is defined by the following expression:

$$P = \rho \cdot g \cdot h \qquad (1)$$

Where: P-hydrostatic pressure; ρ-fluid density; g-gravitational acceleration; h-fluid height.

As fine particles are removed, the permeability of glaciofluvial sediments increases, leading to an acceleration of the progression of this phenomenon, as illustrated in Figure 4.

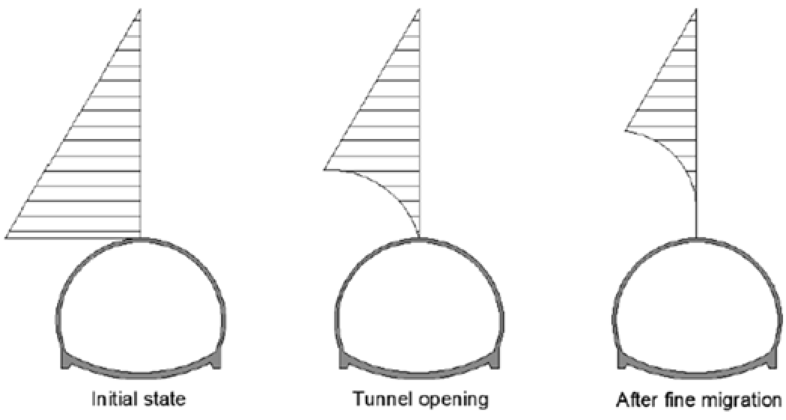

Figure 4. Evolution of pressure gradient during tunnel excavation (drawing by author).

Groundwater infiltration into the tunnel excavation is reduced using grouting treatment. For glaciofluvial sediments, the grouting treatment is designed based on cement or microcement-based mixtures, which not only improve soil properties and ensure tunnel stability during construction but also sufficiently reduce permeability to prevent water ingress.

However, for grouting to be effective, it must penetrate even the smallest voids. This requires the use of ultra-fine suspensions and the highest permissible grouting pressures to achieve maximum penetration and a greater radius of influence. Caution is required when applying high injection pressures in fine-grained materials, as the resulting pressure build-up can induce fines migration.

To execute this procedure correctly, it is essential to understand the properties of the host material into which the grout is to be injected. Consequently, a multi-stage grouting approach is employed, utilizing cement or microcement grouts of varying densities and particle size gradations.

4. PROPOSAL FOR THE REHABILITATION OF THE IBARAC TUNNEL USING THE PRE-GROUTING TECHNIQUE

The collapse that occurred in the Iberac tunnel is of significant scale. The fact that the soil is affected by deformations all the way to the ground surface indicates a lack of soil cohesion. Therefore, soil improvement techniques are needed to form a supporting load bearing ring around the tunnel excavation. Improvement of soil conditions can be achieved by a pre-grouting technique executed from within the tunnel, as shown in Figure 5.

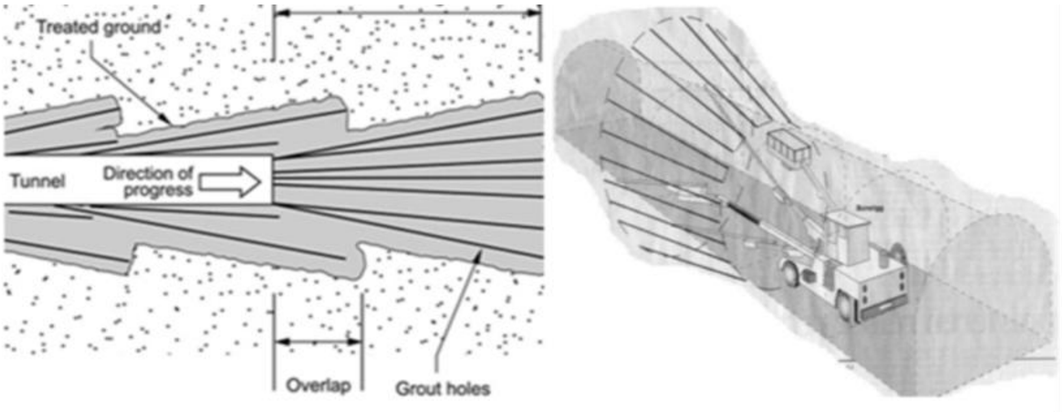

Figure 5. Illustration of pre-grouting in tunnel (drawing by author).

Pre-grouting remains a process where personal experience plays a decisive role in selecting key parameters. Given that the glaciofluvial sediments are predominantly composed of sandy clays, it is necessary to perform pre-grouting at a maximum pressure of 2 MN/m^2; this pressure is maintained for 2 minutes, with a flow that does not exceed 3 l/min. Grouting operations will commence with the drilling of boreholes from chainage km 0+455, progressing successively to km 0+631. These boreholes, with a diameter of 64 mm and a maximum length of 32 m, are to be drilled at an upward inclination of 8%. This geometry ensures that the outer boundary of the grouting zone maintains a 4.5 m offset from the tunnel excavation limit.

All boreholes are monitored in real-time to track pressure, flow rate, and grout volume. Injection is performed using a cement grout with a water-cement (w/c) ratio of 0.5. The pre-grouting is executed in a semi-circular pattern, starting from the lowest borehole and progressing upwards, alternating between the right and left sides. The layout of the 30 boreholes in the crown is illustrated in Figure 6.

A total of 30 boreholes are positioned within the tunnel crown at 2 m intervals. Drilling and grouting are executed in successive 6 meter stages. Following the completion of each stage, the grout is allowed to set before the borehole is re-drilled and extended by an additional 6 m. This incremental process continues to 12 m, 18 m, and so on, until the full design length is reached. A 12 meter overlap is maintained between successive grouting umbrellas. In this configuration, grouting is performed in two primary stages; however, the selection of material (standard or microcement) is determined by the flow rate measured in each borehole upon completion of drilling.

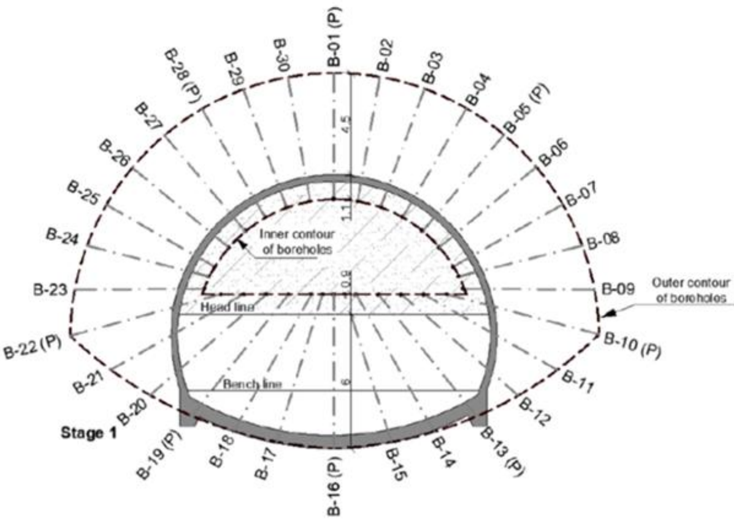

Figure 6. Drilling scheme in the top heading (drawing by author).

Furthermore, a solution is required to homogenize the disturbed soil along the channel that has opened to the surface. It is proposed to perform grouting from the ground surface to stabilize this zone. This approach will facilitate load transfer to the surrounding ground, ensuring the stability and long term durability of the installed support measures. The proposal involves drilling 25 grout boreholes at 3 m intervals, Figure 7.

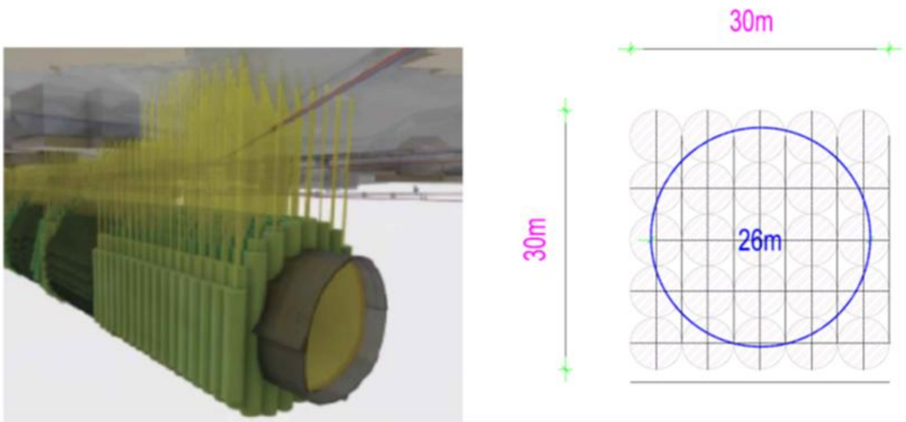

Figure 7. Arrangement of proposed injection boreholes on the surface (drawing by author).

Grouting will commence from a depth of 12 m above the tunnel crown. The cement grout will be injected at a pressure of 5 MN/m^2 to ensure an optimal radius of influence. This process is designed to establish effective bonding between the undisturbed surrounding ground and the disturbed soil zone.

5. CONCLUSION

Experience gained during construction in glaciofluvial sediments has shown that tunneling in such environments involves significant risks. Groundwater influx represents the primary challenge, as it exerts hydraulic pressure and triggers the migration of fines, potentially resulting in structural instability or tunnel collapse.

Immediately after excavation, these sediments may appear stable, allowing for the installation of the primary support; however, this often creates a false sense of security. In glaciofluvial deposits, seasonal increases in groundwater levels cause the leaching of finer particles. The loss of these particles destabilizes the soil skeleton and leads to a rapid increase in ground pressure, causing the deformation of the existing primary support. In sections where the support is already installed, sudden deformations may occur due to this phenomenon. The magnitude of these deformations is illustrated by the collapse at the Ibarac tunnel (km 0+473), where deformations reached the ground surface despite an overburden exceeding 100 m.

Following the failure, the formation of a continuous load-bearing ring around the tunnel opening became unreliable. Additionally, the ongoing threat of internal erosion (leaching of fine particles) persists throughout the tunnel's operational lifespan. Since this process undermines long-term stability, extensive collapses should be addressed through systematic pre-injection to stabilize the surrounding ground.

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