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Geotechnical Challenges and Adaptive Excavation Strategies: The Konovo Tunnel Case Study

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Abstract

The Konovo Tunnel is an approximately 500 m long single-tube road tunnel located on the Third Development Axis expressway in Slovenia, along the section between Velenje and Slovenj Gradec. Excavation was carried out under an overburden of up to 50 m in highly heterogeneous geological conditions, consisting of tectonically intensely fractured dolomite, sandy marl with embedded dolomite blocks, marlstone, claystone, and locally occurring coal seams. Despite its relatively short length, the tunnel construction required the management of several significant geotechnical challenges. One of the key challenges was a continuous section of marlstone and claystone forming a relatively homogeneous geological sequence beneath two residential buildings, with an overburden of approximately 35 m. In this zone, a technologically adapted phased excavation scheme was implemented to ensure adequate control of surface settlements and to maintain the structural safety of the overlying buildings. Additional critical challenges were encountered in the portal areas. The northern portal intersected a 58 m long section located within a slope characterized by very low global stability. To mitigate slope instability risks and ensure a safe transition to underground excavation, a pile-supported cut-and-cover construction method was adopted.At the southern portal, an atypical constraint required the tunnel to be excavated and broken through under extremely shallow overburden due to permitting limitations. The breakthrough was therefore carried out following the excavation of a southern pre-cut, which enabled controlled exposure of the tunnel face.Although relatively short in length, the Konovo Tunnel project encompassed a wide range of geotechnical and construction challenges, which were successfully managed through continuous geological and geotechnical monitoring, as well as adaptive modification of excavation and support measures during construction.

Keywords

Tunnelling, Heterogenous conditions, Low overburden, Ground movement control, Geotechnical supervision.

1 Introduction

1.1 Tunnel and portal layouts

The new road connection from Velenje South to the Slovenj Gradec south interchange has a total length of 17.5 km. One of its key elements is the 1.9 km long section between Konovo and Škale, which comprises a viaduct and the approximately 500 m long Konovo Tunnel. The Konovo Tunnel is designed as a single-tube, bidirectional road tunnel with a design speed of 70 km/h. The tunnel cross-section consists of two traffic lanes, each 3.25 m wide, with a central separation strip 0.50 m wide and two edge strips 0.35 m wide, extending to the service walkway. The overall excavation profile of the tunnels was approximately 100 m2, while the primary support was carried out using the postulates of the NATM method (Rabcevicz, 1964). The overall tunnel structure includes a 425 m long tunnel tube, a 58 m long northern portal structure, and a 14 m long southern portal structure, resulting in a total tunnel length of 497 m.

The northern portal, shown in Fig. 1 is designed as a covered cut-and-cover structure. Due to extremely poor geotechnical conditions, the construction of a sufficiently wide-open excavation for the transition into the tunnel section was not feasible. The ground conditions in this area consist primarily of high plasticity clays containing completely disintegrated dolomite and lenses of organic material, resulting in very low stability of the existing natural slopes. Portal structure was designed as a covered trench gallery (cut and cover structure) using the top-down construction method. Initially, bored piles were installed along the tunnel alignment, followed by the construction of the upper and lower bracing slabs, which together formed the structural frame of the gallery. After completion of the structural elements, backfilling above the top slab was carried out, followed by measures for surface water management, prior to the excavation beneath the top bracing slab.

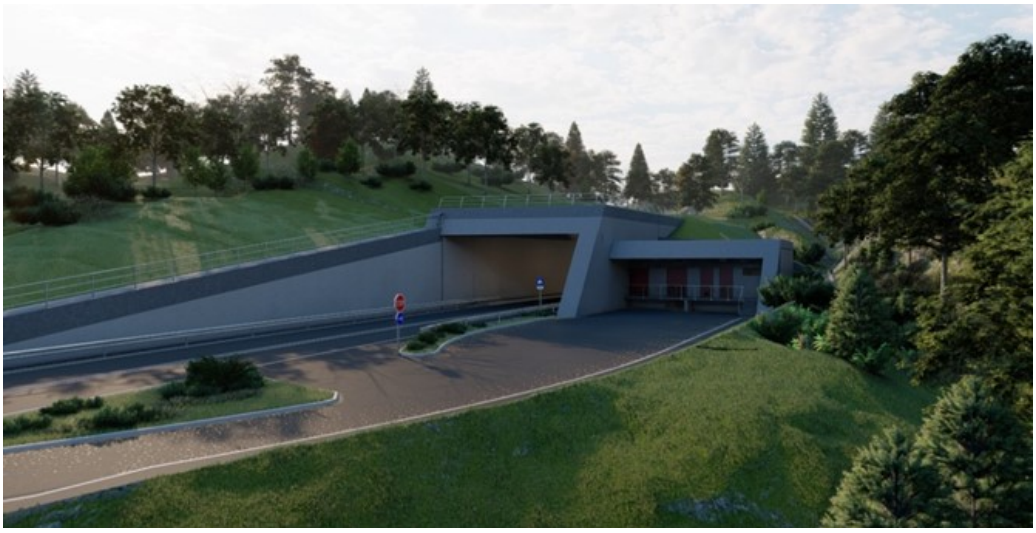


Fig. 1 Render visualization of the final design of the northern portal of the Konovo tunnel.

A specific feature of the southern portal is the transition of the tunnel to a viaduct located on a very steep slope, with the portal positioned relatively high on the hillside. A conventional tunnel portal configuration, combined with reinforced earth embankments combined with a stone masonry retaining structure was executed in limited spatial conditions.

2 Geological and hydrogeological conditions

2.1 Geological and hydrogeological observation during construction

Geotechnical model based on site investigation carried out in 2019 predicted at northern portal a presence of grey to brown high plasticity clays with interlayers of sandy clay, silt, and clayey fine sand. Clays transitioned into organic clays locally with coal lenses reaching thicknesses of several meters. The bedrock along the tunnel alignment was interpreted as Lower Scythian light grey to yellowish dolomite, with interlayers of siltstone and locally marlstone. Siltstone layers occur mostly in frequent alternation with dolomite. At the southern portal, layers of marly limestone and siltstone were also identified. The rock mass was expected to be strongly fractured along the tunnel routewhile

locally tectonically crushed to soil-like material within the tectonic gouge as seen at the northern portal.

During the excavation of the portal cuts and the tunnel, continuous geological and hydrogeological monitoring was carried out. Geological monitoring comprised surface inspections, logging of piles and boreholes, and detailed documentation of the excavation, including face mapping, bench, and invert inspections. Hydrogeological monitoring included the measurement of groundwater inflows within the tunnel, monitoring of piezometric levels at the surface, and the interpretation of the changing hydrogeological conditions during construction.

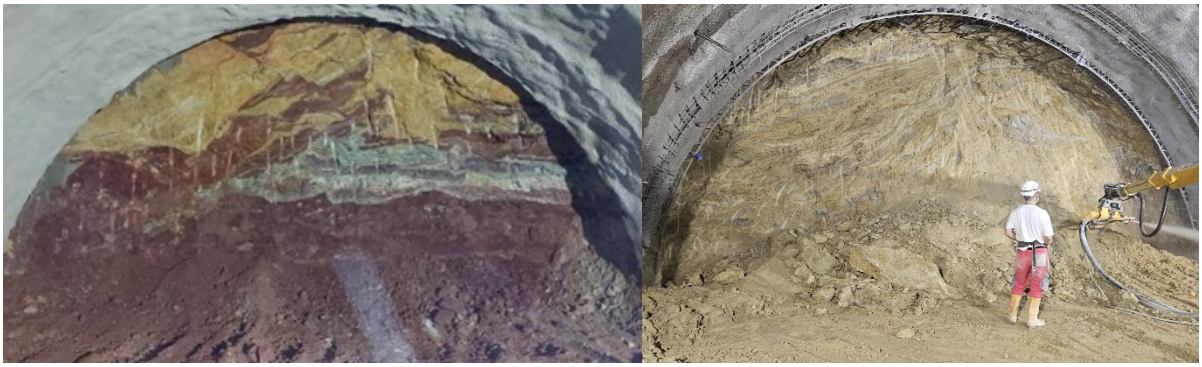


Fig. 2 Comparison of tunnel excavation faces in red and yellow fine-grained marls (left) and in tectonically fractured dolomite with interbedded layers of sandy marl (right).

Comparison between the predicted and observed conditions generally showed that the dolomite unit did not occur as a continuous and homogeneous strata, but in the form of individual blocks tectonically embedded within alternating layers of red, brown, and grey siltstone. In addition, a section of the tunnel was encountered where the excavation consisted almost entirely of red, brown, and grey siltstone layers, shown in Fig. 2, which was reclassified as a separate unit of weak rock. Excavation of the tunnel revealed that this unit dominates in several tunnel sections and typically forms entirely the bench and invert zones. In general, geological mapping repeatedly revealed intense tectonic deformation of the rock mass along the tunnel layout, where lithological conditions change rapidly along fault zones.

Comparison of the predicted and observed hydrogeological conditions showed good agreement with water appearing mainly in the form of seepage, dripping, and minor inflows at the tunnel invert. In the initial section of the tunnel starting from the northern side, the inflows of up 3 l/min per 25 m of tunnel layout were detected. In the mid tunnel section, initial inflows of 30–45 l/min per 10 m of tunnel were recorded during the excavation of the top heading while the steady-state inflows later decreased to approximately 2.5 l/min per 100 m of tunnel. This reduction can be attributed to the effective drainage of groundwater during excavation, which might have induced settlement of the ground surface, as it will be commented later.

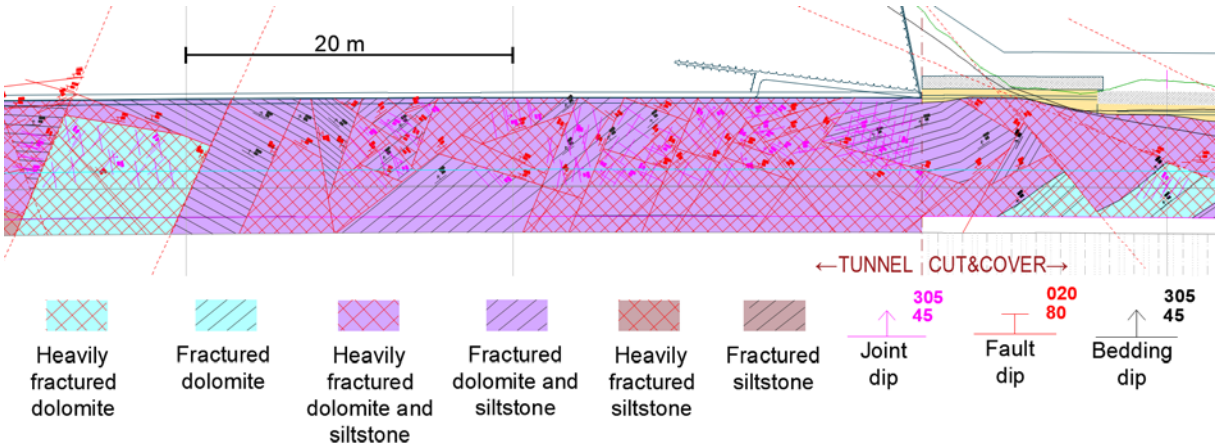


Fig. 3 Longitudinal geological profile of the northern portal area, illustrating tectonic deformation and geological heterogeneity.

3 Challenges encountered during tunnel construction

3.1 Tunnel excavation and support

The transition from the cut and cover structure to the tunnel excavation at the northern side was carried out using a pipe roof support system. This was necessary to address the abrupt transition from the portal structure to the underground excavation, combined with very low overburden, continuous groundwater inflows from nearby torrential gullies, and rapidly varying geological conditions (

Fig. 3).

During the subsequent tunnel excavation, the geological conditions continued to change frequently, requiring permanent adjustment of excavation step lengths and support types. Despite the well-known shortcomings (Kovari, 1994), the flexibility of NATM (Karakus & Fowell, 2004) was found particularly suitable for these geological conditions. As part of the optimisation of the support system the reinforced shotcrete lining was replaced by the steel fibre shotcrete. However, it soon became evident that the wet sandy siltstone base combined with embedded dolomite blocks did not provide suitable conditions for this optimisation. The steel fibre shotcrete exhibited poor adhesion to the substrate, resulting in detachment during application and significant loss of material. After a short trial section, steel fibre shotcrete was abandoned. Following similar conditions experienced in Karavanke tunnel (Jovičić & Volk, 2022), this case also confirmed that wet rock mass substrates with a tendency for disintegration or composed of fine-grained material are not suitable for the application of steel fibre shotcrete based on conventional mix designs.

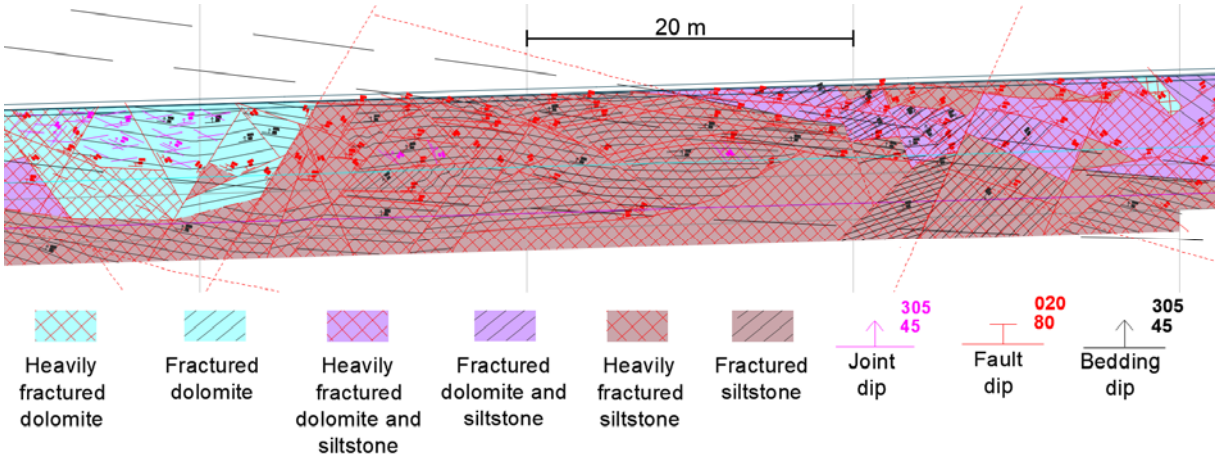


Fig. 4 Geological profile of the tunnel featuring a red shale marl section extending across the entire tunnel cross-section.

3.2 Ground deformation and surface settlements

Beyond initial section, the tunnel entered a relatively homogeneous geological zone (Fig. 4) composed predominantly of red shaly siltstone. From a geotechnical perspective, the ground conditions became significantly less favourable so that the increase in tunnel displacements began to manifest at the ground surface. Two existing residential buildings, which were located directly above the tunnel alignment (Fig. 5) so that a precise levelling monitoring system was established on both structures to measure settlement and structural movements during the tunnel excavation.

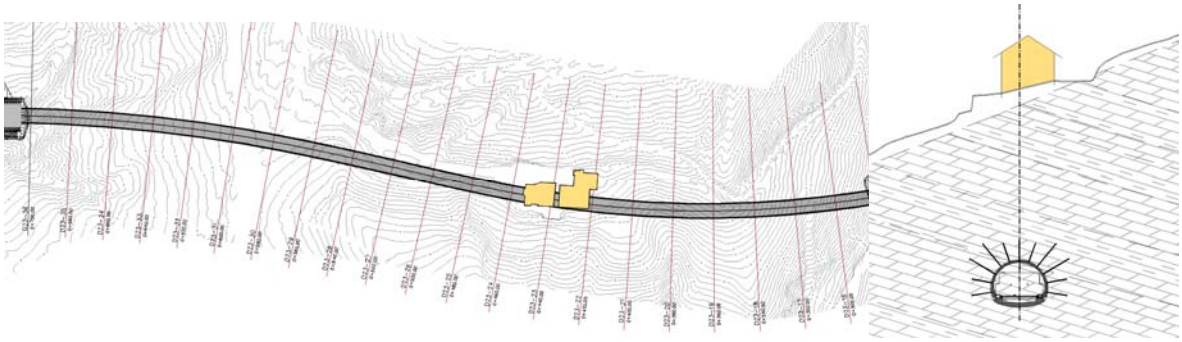


Fig. 5 Location of the 2 residential buildings, situated 35 m directly above the tunnel alignment.

The new support type was introduced in the tunnel approximately 15 m before entering the red siltstone zone within the zone of influence on the settlements of the buildings. The support measures consisted of 20 cm thick shotcrete reinforced with a single layer of Q189 steel mesh, combined with 4 m long SN rock bolts of nominal capacity of 250KN, installed in a 2.0 m grid, with an excavation step length of 1.7 m. Due to increase in settlements recorded on the buildings above the tunnel, the excavation face was temporarily stopped on few occasions, and the invert was closed all the way to the top heading. In continuation, a "stop-and-go" construction procedure was used to limit further deformations.

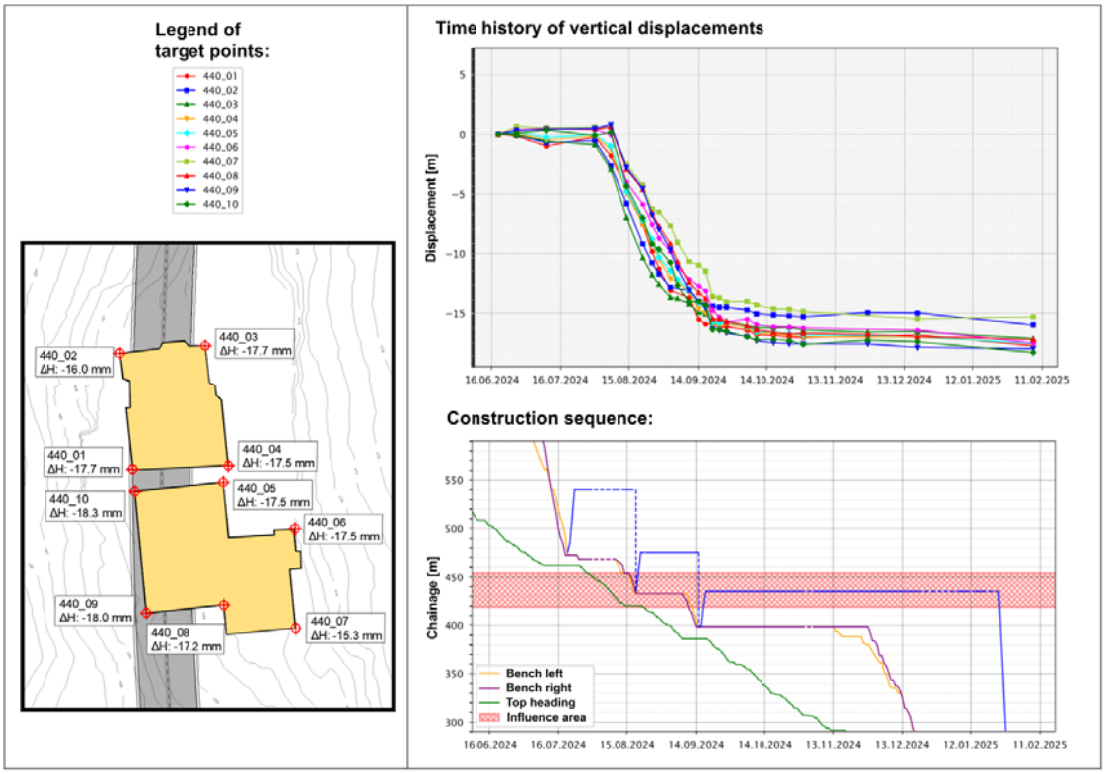


Fig. 6 Presentation of final precise levelling measurements at residential buildings above the tunnel alignment relative to the progress of the tunnel excavation (legend to the left shows the names of different target points at the surface and the buildings).

Mostly vertical tunnel displacements reached approximately a total of 15 cm after completion of the excavation, while surface settlements were up to 18 mm (Fig. 6) where differential settlements on individual structures remained limited to approximately 1.5 mm and no damage to any of the buildings was detected. The absence of differential settlements indicated that lowering of groundwater table may have a significant contribution to this settlement trough.

3.3 Local instabilities at the face of the excavation

The next major challenge encountered during the construction of the Konovo Tunnel was a local instability zone within geological conditions dominated by highly fractured and tectonically disturbed dolomite in the approach to southern portal area. During the excavation, the rock mass proved friable,



Fig. 7 Block failure of dolomite during the excavation of the top heading in the low overburden zone.

while locally larger dolomite blocks detached along unfavourably oriented discontinuities, causing local collapses, as illustrated in Fig. 7. When advancing with an excavation step of 1.3 m, excessive block detachment from the crown occurred, resulting in the formation of larger voids. Considering the low overburden of approximately 6 m above the tunnel axis, and the potential risk to the global stability of the slope caused by ongoing block detachment, a 9 m long pipe umbrella consisting of 26 pipes to mitigate the threat of local instabilities that could potentially have developed into larger-scale collapses or even global tunnel instability at this section. Excavation beneath the pipe roof was carried out with a 1 m excavation step, combined with relatively stiff support measures. These included 25 cm thick shotcrete reinforced with two layers of Q283 steel mesh, together with 6 m long IBO self-drilling anchors installed in a 1.5 m grid. Face stability was ensured using a temporary support core and shotcrete protection. After installation of the primary support, the cavities were filled by pressure grouting to ensure proper contact between the lining and the surrounding ground.

4 Conclusions

Despite its relatively short length of approximately 500m, Konovo tunnel presented a wide range of geotechnical challenges, typical of variable geological conditions and shallow urban tunnelling. One of the key challenges was the construction of the northern portal gallery, which had to be executed as a top-down, cut-and-cover structure due to demanding geological conditions. Another important factor during construction was the strict limitation of ground movements beneath residential buildings, which required continuous adjustment of the excavation sequence and careful implementation of the primary support measures. A particular difficulty occurred during the approach to the southern portal, where detaching blocks of unbound dolomite posed a threat of local instabilities that could potentially have developed into larger-scale collapses or even global tunnel instability. These risks were successfully mitigated through the appropriate selection and timely implementation of additional support measures.

Despite the challenges described, the construction process was efficient and largely consistent with expectations. It should be noted, however, that tunnel construction was constrained by a regulatory restriction prohibiting night-time work in urban areas. Such a limitation is uncommon in modern tunnelling practice, particularly under demanding geotechnical conditions, where continuous operations are often essential. Restricting work to daytime operations significantly reduced excavation progress, since only temporary stabilisation measures could be implemented, thereby negatively affecting tunnel stability and the overall construction schedule.

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