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Numerical models for the design and construction of new underground structures at CERN (HL-LHC), Point 5

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ABSTRACT: The Large Hadron Collider (LHC) is the latest, most powerful, world's largest underground particle accelerator realized on the CERN site. High-Luminosity LHC (HL-LHC) is a new project aimed to upgrade the LHC at Point 1 (ATLAS in Switzerland) and Point 5 (CMS in France) to enhance scientific progress. This paper describes the design and construction issues developed at Point 5 for the new underground structures near the LHC tunnel. The project requires new technical infrastructure: an additional shaft with a 12 m-diameter and 60 m-height, cavern with 270 m^2 cross-section, approximately 500 meters of galleries connected to the LHC tunnel, vertical linkage cores and additional technical buildings at the surface. This site's geological ground model lies in an area covered by Quaternary moraine with two independent aquifers. The bedrock of Molasse comprises sub-horizontal lenses of heterogeneous sedimentary rock, that is known to locally retain hydrocarbons and to have a swelling behaviour. To investigate the heterogeneous behaviour of the rock mass composed of several layers with different strengths, numerical calculations have been performed, under a 2D plane strain condition with RS2 9.0 FEM-software. The purpose of using the software was to design both the rock-supports and the concrete inner lining for the galleries and the shaft. Data from a comprehensive monitoring system with pre-defined threshold values was compared to the 2D FEM results, confirming the importance of the observational method to verify the assumptions used in the numerical modelling.
The execution of the underground works started in April 2018. The excavation of the main underground works has been successfully completed without any critical impact on the nearby existing underground structures. The completion of the works is scheduled for September 2022.

Keywords: Finite element method, numerical analysis, rock mechanics, observational method

1 INTRODUCTION

The European Organization for Nuclear Research (CERN) is an organization with 23 member states worldwide, and its headquarter is based in Geneva. The Large Hadron Collider (LHC) is the world's largest underground particle accelerator placed on both sides of the Swiss-French border. The data collected by this unique instrument in the world has allowed the discovery of the Higgs boson in 2012.

The High-Luminosity LHC is a new project aimed at enhancing the LHC experiments, in order to produce more data by increasing the number of particle collisions by a factor of 10. This project will be operational in 2026 and requires new technical infrastructure for the two main detectors, respectively, at Point 1 (ATLAS) and Point 5 (CMS), as shown in Fig.1.
At Point 5, the new HL-LHC underground structures are placed on the inner side of the existing LHC ring at an average distance of approximately 50 m from the LHC axis and located 7 m above the level of the existing LHC tunnel crown. The new underground structures consist of the following main objects: i) a new shaft PM57 12 m diameter and 60 m deep with at the base a service cavern US57/UW57 of excavation area 270 m^2, ii) a power converter gallery UR55 with an excavation area of approximately 60 m^2, iii) two pairs of service galleries UA57 /UA53 and UL57/UL53 with excavation areas of 45 m^2 and 20 m^2 respectively, iv) 16 vertical linkage cores to the existing LHC

1.7 m excavation diameter and 5m deep, v) 2 personnel escape galleries UPR53/UPR57 of excavation areas 25 m^2 (Fig.1, right). All the new underground structures are designed with a double lining system. For temporary support, a design working life of 10 years is required. For the final lining and the waterproofing system, a design working life of 100 years is needed according to the specification. Additional technical buildings at the surface are currently under construction.

The main challenges of the construction project (Canzoneri et al., 2019) were related to the following: i) the criteria against the vibrations and settlements induced from the excavation works on the existing structures where experiments were in progress; ii) the excavation inside a heterogeneous rock mass with known swelling behaviour and containing hydrocarbons.

This paper describes the key role of the RS2 9.0 software, which made it possible to overcome parts of these challenges.

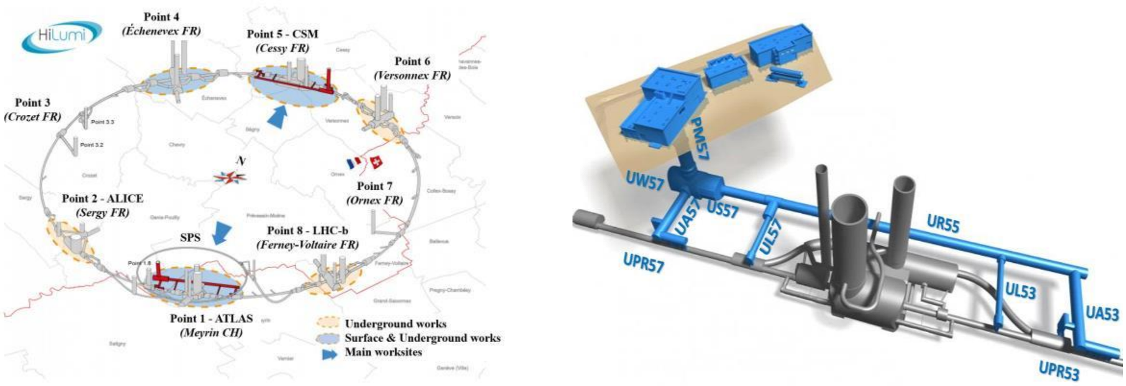

Fig. 1 – Left: HL-LHC project; Right: new (in blue) and existing (in grey) structures at Point 5 (courtesy CERN).

2 GEOLOGICAL CONDITIONS

The general geological profile at the site of Point 5 as shown in Fig. 2 can be described, from surface downward, as follows: i) a limited thickness of fill put in place during the previous works comprising of colluvial soils; ii) fluvial-glacial soils resting on heterogeneous glacial (Wurmian) moraines, these deposits have an overall thickness ranging from approximately 30 m to 50 m; iii) underlying the moraine deposits, the rock formation of the red Molasse (Chattian age, Tertiary).

The new underground structures have an overburden of approximatively 60 m. These are excavated inside the Molasse unit, except the upper part of the PM57 shaft which extends to a depth of approximately 22 meters, crossing the soil units described in Table 1. Available ground investigation data from previous underground works showed that the Molasse in this area is a highly heterogeneous rock mass known to locally contain liquid and gaseous hydrocarbons (Kurzweil, 2004). The Molasse layers are usually 0.5 to 5 m thick, and their stiffness can vary significantly from one layer to another. Three main units can be distinguished: i) marls: the finest grain-size unit of the molasse, characterized by a high percentage of clay minerals, including the swelling minerals illite and smectite; ii) sandstones: characterized by coarser grain-sizes of silt and fine sand, in approximately similar percentages, cemented by calcareous cement; iii) transitional materials between marls and sandstones.

Taking into account the information from the previous underground excavations and the recent ground investigation, the different facies were modelled (Table 2) in the software RocLab 5.0. The in situ-stress was determined from the in-situ investigations where the K_0 varied between 1.25 and 2 for the rocks and the K_0 was 0.7 for the soils. In terms of hydrogeology, two aquifers were identified: i) upper aquifer, phreatic, located within the fluvio-glacial soils; ii) lower aquifer, within the underlying moraine. The two aquifers do not communicate, being separated by the less permeable layers, though tracing tests carried out in the Point 5 site showed local connections, natural or maybe due to anthropic activities. As the permeability of the Molasse obtained from in situ tests was very low ($k < 10^{-7}$ m/s), the rock mass was considered to be impermeable.

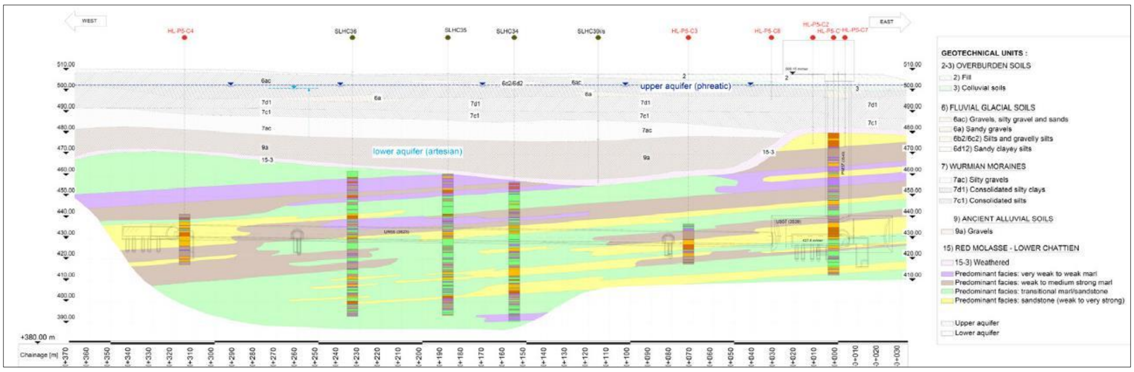

Fig. 2 - Geological profile at Point 5.

	γ [kN/m³]	OCR [-]	ϕ [°]	ψ [°]	c' [kPa]	c_u [kPa]	E [MPa]	υ [-]	k [m/s]
Fill	21.0	-	28-30	8	0	-	40-120	0.3	-
Colluvial soil	19.5	-	27-29	7	5-10	> 40	>100	0.3	$4{\cdot}10^{-8}\div 8{\cdot}10^{-7}$
Silty gravel	23.5	-	32-36	12	0	-	200-450	0.3	$9{\cdot}10^{-4}$
Sandy silt	22.5	2	32-34	12	10-15	150-250	100-170	0.3	$9{\cdot}10^{-8}\div 1{\cdot}10^{-7}$
Silty clay	22.5	2	30-32	10	10-15	100-200	70-140	0.3	$(4\div 9)\cdot 10^{-8}$

Tab. 1 - Geotechnical parameters for soil units.

	γ [kN/m³]	ϕ [°]	ψ [°]	ϕ_{res} [°]	c [MPa]	c_{res} [MPa]	ϕ_{res} [°]	σ_c [MPa]	σ_t [MPa]	E_{mc} [MPa]	E_{md} [MPa]	υ [-]	k [m/s]
Very weak to weak marls	24.0	18.5	0	17.0	1.2	0.5	17.0	3.2	0.5	500	850	0.30	$1{\cdot}10^{-7}$
Weak to strong marls	24.5	30.0	10	30.0	2.6	1.3	30.0	9.4	1.0	1260	2500	0.30	$1{\cdot}10^{-7}$
Marl/sandstone	25.0	38.0	18	34.0	4.8	1.6	34.0	20.0	1.8	3000	4500	0.30	$1{\cdot}10^{-7}$
Sandstone	23.0	41.0	21	34.0	2.1	1.0	34.0	9.1	0.6	1500	2500	0.30	$1{\cdot}10^{-7}$

Tab. 2 - Geotechnical parameters for rock units.

3 THE ROLE OF NUMERICAL ANALYSES DURING THE DESIGN PHASE

In the first design phase, preliminary calculations were carried out using the software RocSupport 4.0 for an initial assessment of the required rock supports and evaluating the stress-strain behaviour. The Convergence-Confinement analyses were performed according to the Duncan Fama (1993) solution based on the different values of the geomechanical parameters shown in Table 2. From these analyses, the behaviour of the rock masses was generally determined to be within the elastic range, generating a total maximum tunnel displacement without support of less than 1 cm. Only by considering a worst-case scenario with a section entirely within the unit of very weak marls were a total maximum displacement of 5 cm with a plastic radius of approximatively 3 m calculated. Analyses using the UnWedge 4.0 software were carried out for the stability analyses of the rock wedges to design the rock bolt pattern.

FE 2D analysis, under plane strain conditions, was performed using RS2 9.0 software to investigate the rock mass's heterogeneous behaviour composed of several layers with different strengths. The results of these analyses have: i) provided the design of the rock- supports; ii) assessed the potential impact on the existing underground structures; iii) provided the design of the permanent concrete inner lining.

Numerous calculation models were created to design the various standard cross-sections and rock support classes required according to the critical narrow tunnel geometric configurations and different scenarios for the location and thickness of the molasse layers. For all models, an elasto-perfectly plastic Mohr-Coulomb criterion was adopted to reproduce the soil behaviour; for the rock mass, the equivalent Mohr-Coulomb with peak and residual strength parameters were adopted (Table 2). The external dimensions of the FE models were chosen to minimize boundary effects. Boundary conditions consist of fixed horizontal/vertical displacements along x-direction (hinge) and fixed vertical displacements along y-direction (roller). The domain has been discretized through triangular meshes, increasing the discretization around the underground structures. To reproduce the excavation phases, the convergence-confinement method (Panet and Guenot, 1982) was adopted with the stress reduction λ evaluated based on the ground reaction curve and calibrated based on the monitoring results from the previous underground excavations. Bolts were modelled as a fully bounded element or Swellex type, and the shotcrete liner was assigned as a standard beam. The permanent concrete inner lining is also considered a standard beam but with a selected composite liner function.

Figure 3 shows the numerical model for evaluating the influences between the existing USC55 cavern (width of 19 m, height of 17 m and length of 85 m) and UR55 tunnel. The UR55 was modelled with support class 1 consisting of: friction bolts L = 4m $F_{tk} \geq 240$kN (distance 1.5m / 1.5m), 1st layer 5 cm thickness of fibre reinforced shotcrete C20/25, wire mesh 1.89 cm^2/m and 2nd layer 10 cm of shotcrete C20/25. For the USC55 cavern, rock supports were modelled using shotcrete C25/30 with total thickness of 30 cm, rock bolts $F_{tk} \geq 600$ kN L = 9 m (transversal distance 0.85 - 1.5 m x longitudinal distance 1.5 m) and rock bolts $F_{tk} \geq 400$ kN, L = 6.00 m (transversal distance 1.5 m x longitudinal distance 1.5 m). The inner lining of the US55 cavern comprises reinforced concrete C40/50 with a thickness of 57 cm. From the analysis of the results, the following main conclusions were identified: i) the behaviour of rock masses was mainly elastic with narrowed plastic zones that were not in contact with the nearby tunnels; ii) the increases in stresses and displacements of the rock masses around the existing tunnels were low; iii) the increase of displacements in the existing crown cavern were estimated to be a maximum of 3 mm and in the tunnel invert to be a maximum of 1 mm, all below threshold values for the ongoing CERN tests; iv) the increase in axial forces, bending moments and shear forces on the existing tunnel lining were within the ULS and SLS criteria.

Figure 4 shows the model used to evaluate the excavation of the new UA57 tunnel which is located at a minimum distance of approximately 5.5 m above CERN's main ring (R57). For the existing R57, the lining was modelled as a standard beam of concrete C25/30 without reinforcement with a thickness of 22 cm. For the UA57, the rock support class 2 was modelled comprising of: shotcrete C20/25 with total thickness of 25 cm, lattice girders 3G 70/20/26 with a spacing of 1.2 m and radial fully grouted bolts L = 4.00 m, $F_{tk} \geq 250$ kN with a spacing in plane of 1.50 and out of plane of 1.20 m. This model also confirmed a predominantly elastic rock mass behaviour with narrowed plastic zones that were not in contact with the nearby tunnels. The increase in displacements in the existing tunnel was estimated to be a maximum of 1 mm. From these analyses, it was concluded that no additional supports were necessary for the existing LHC.

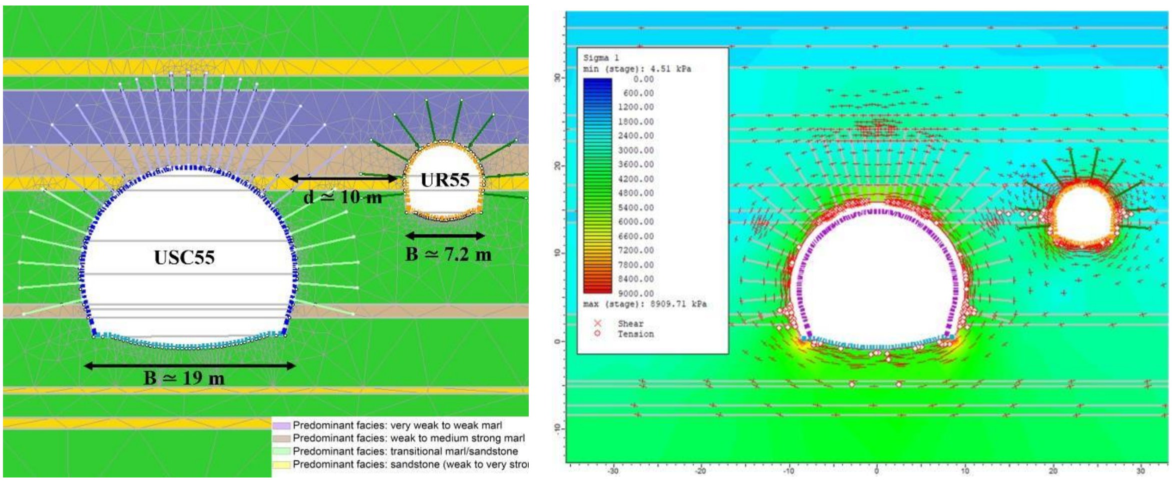

Fig. 3 - Model for the evaluation of UR55 tunnel excavation on the existing UCS55 cavern. Left: FE model geometry and rock units; Right: stress and yielded zones on the rock mass.

To evaluate the sections near the several crossing zones where 3D effects are critical, the same conservative geotechnical parameters were adopted as those used in the models analyzed using RS2 2D software. The tunnel inner lining was reinforced only in the crossing zones as determined from the results of the 2D numerical models. Stresses acting on the final supports were compared with those obtained from simplified beam-spring 2D models also taking into account the loads induced by temperature, swelling and creep effects according to the load combinations stated in Eurocode.

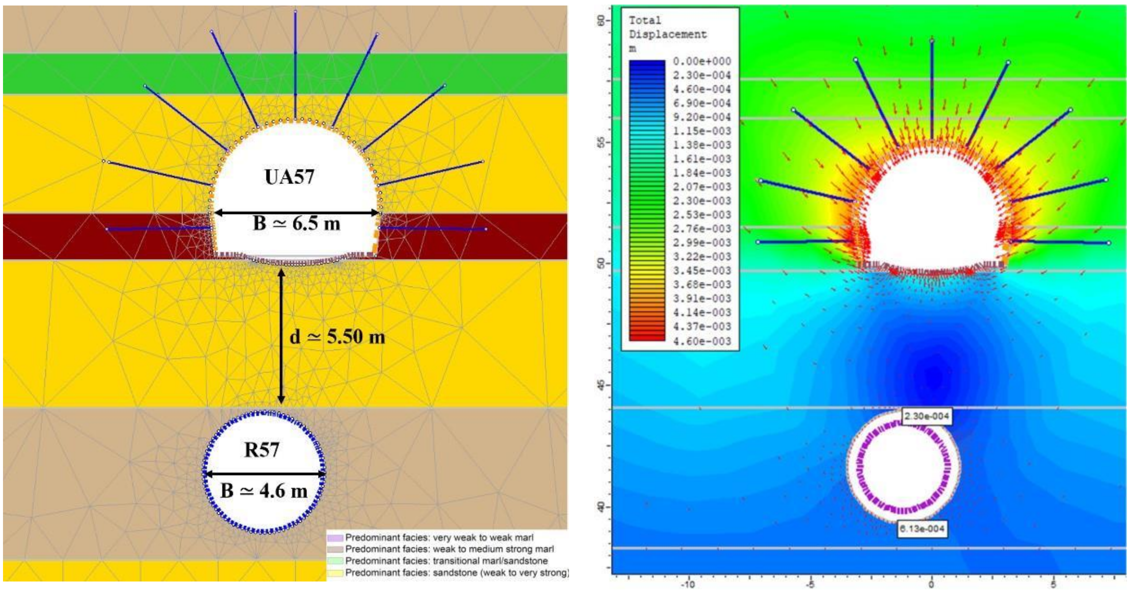

Fig. 4 - Model for the evaluation of UA57 tunnel excavation on the existing LHC. Left: FE model geometry and rock units; Right: total displacements resulting from the excavation of UA57.

4 THE ROLE OF NUMERICAL ANALYSES DURING THE CONSTRUCTION PHASE DATA

A comprehensive monitoring system with predefined threshold values was compared to the 2D FEM results, confirming the importance of the observational method to verify the assumptions used in the numerical modelling. Convergence monitored for the new underground structures were a maximum 1 cm and those for the existing structures, were on the scale of a millimeter.

Considering the actual ground conditions encountered during the excavation stage, some optimizations of the rock supports were possible. Figure 5 shows the back-analysis model for the UR55 rock support class 1 that was performed to assess the lower rock bolts' removal, thus allowing an increase in excavation rate and a reduction in project costs. The models' results showed that the structural safety was still within threshold values and that the effects on the nearby structures were similar to those in the original design solution.

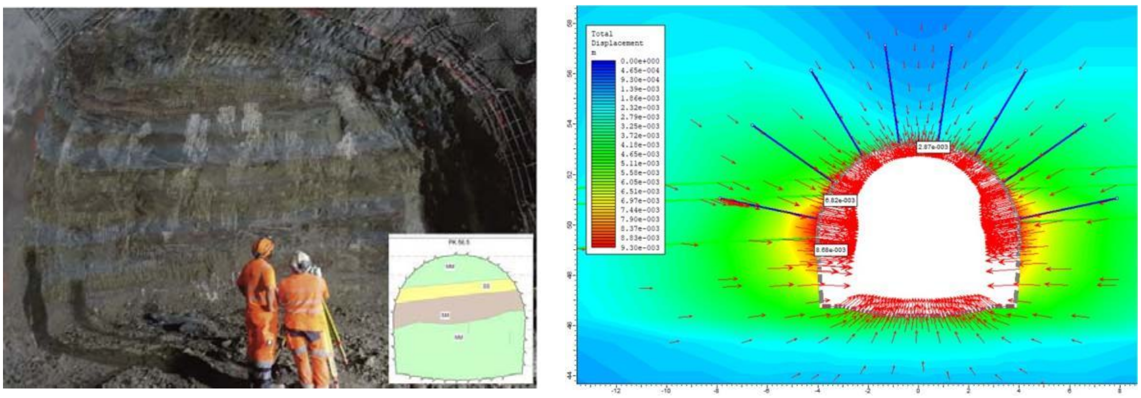

Fig. 5 - Left: convergence monitoring. Right: back-analysis for the optimization of rock supports.

Another model was done at the PM57 shaft, where it was also possible to optimize the reinforced concrete inner lining. Based on the actual ground conditions encountered and the monitoring data, FE axial-symmetric models were created, including all the effective excavation phases (Figure 6). The rock pressure along the depth of the shaft was compared to those estimated from the Convergence-Confinement analysis. The re-evaluation led to a reduction of the rock load acting on the final lining from 520 kPa to 400 kPa. Horizontal and vertical steel reinforcements were re-evaluated using Ø14 mm with 150 mm spacing saving a significant amount of steel (about 40%) while keeping the thickness of the inner lining (60 cm) and the type of concrete used (C35/45) unchanged.

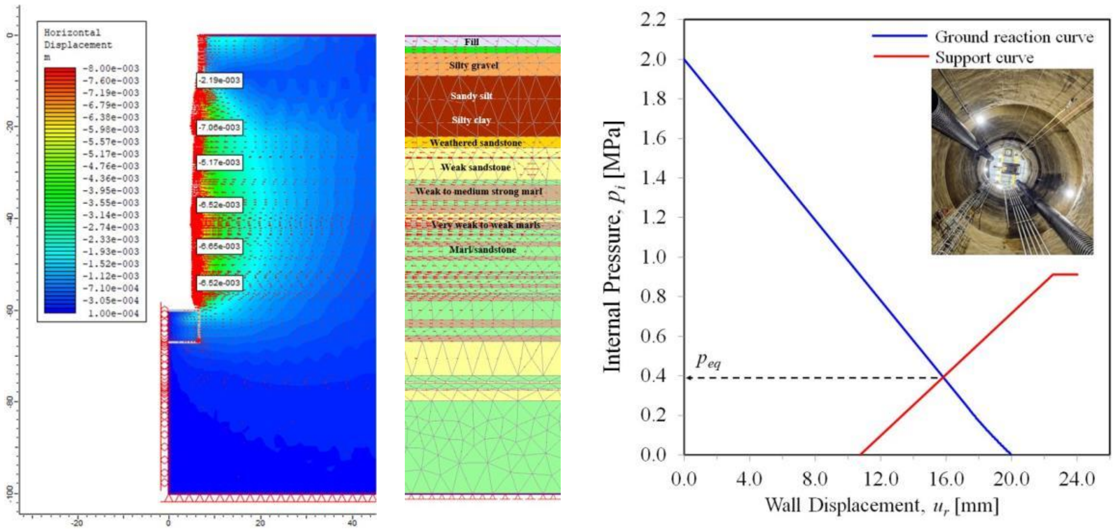

Fig. 6 - Back-analysis of Shaft PM57. Left: axial-symmetric FE model showing horizontal displacement calculated from the cross-check geology; Right: ground reaction and rock-support curves obtained from RocSupport 4.0.

CONCLUSIONS

In this paper, the benefits of using a high-performance and reliable calculation FEM software both in the design and the construction phases in conjunction with a comprehensive monitoring system installation are presented. Numerical modelling was a fundamental tool that simulates numerous excavation phases in complex ground conditions and evaluates the potential impact of the excavation works on nearby underground structures. The excavation of the main underground works was successfully completed without any critical impact recorded on the nearby existing underground structures. Comparisons with continuous monitoring data allowed for optimizing the overall design of supports, allowing an increase in excavation rate and a reduction in project costs. The execution of the underground works started in April 2018, and currently, the construction phase is on track with the contractual Construction Programme. The completion of the works is scheduled for September 2022.

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