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Effect of the Distribution of Joints in the Tunnel Excavation Phase Simulation: A Case Study

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Abstract. This work aims to show the effect of joint distribution taking into account during the excavation phase simulation of the tunnel, in our case, Tunnel 4 of Kherrata (Algeria). At this effect, a comparison is made between the simulation of a sequential excavation phase using the Finite Element Method (FEM) with an equivalent model and a simulation that incorporates the presence of the sets of joints in the Model. The first simulation represents an equivalent model representing the whole rock mass with the Hoek and Brown Generalized criterion. Conversely, a Discrete Fracture Network (DFN) combined with the finite element method represents the distribution of the sets of joints used in the second Model. In our case study, the geomechanical classification of the rock mass, carried out using the Rock Mass Rating (RMR) and the Rock Mass Quality (Q) system methods, with values of 33 and 1.06, respectively, indicates that the south portal zone of Tunnel 4 exhibits poor quality. To assess the stability of the tunnel during the excavation phase, we conducted a numerical simulation using two models: an equivalent model and a second model that took into account the distribution of discontinuities. Finally, the results indicate that the presence of joints has a significant impact on the simulation outcomes.

Keywords: Tunnel; finite element method; discrete fracture network; rock mass; joints.

INTRODUCTION

Analyzing the stability of underground structures excavated in fractured rock masses remains a significant challenge in research and geotechnical engineering. The rock mass's complex, anisotropic, and discontinuous nature complicates this assessment, primarily due to the influence of discontinuities. Several methods are available for evaluating the stability of rocky formations, taking into account these factors. These methods can be categorized into semi-empirical approaches, such as the Rock Mass Rating (RMR) and Q-Barton, as well as numerical methods.

Rock mass classification is designed to analyze data related to the properties of rock materials, their features, and the geometry of excavations in order to derive representative discontinuity parameters, thereby supporting informed and logical decisions in rock engineering (Boukarm et al., 2017). Numerous geomechanical classification systems exist, such as Terzaghi's Rock Load Theory (1946), Rock Quality Designation (RQD, 1967), Rock Mass Rating (RMR, 1973), Rock Mass Quality (Q-Barton, 1974), and the Geological Strength Index (GSI, 1997), among others.

These approaches have been fundamental in the field of underground construction. However, since these systems are largely semi-empirical, their results require validation through numerical modeling.

Numerical methods in continuum mechanics are extensively employed in geotechnical engineering. The introduction of this approach to rock mass modeling relies on the equivalent medium method, which assumes the rock masses behave as a continuum but does not consider the spatial distribution of joints. In contrast, the Discrete Fracture Network (DFN) method enables the generation of statistically representative joint models, significantly improving alignment with real-world geological conditions. A hybrid methodology integrating the Finite Element Method (FEM) with DFN techniques offers synergistic advantages by combining FEM's computational efficiency in continuum analysis with DFN's capacity to represent discrete fracture systems.

The DFN is a discrete model developed in 1970, mainly to simulate the flow of fluids and transport processes through connected rock fractures. Discrete fractures are defined in a deterministic or stochastic way. Dershowitz and Einstein (1988) presented a comprehensive work that brings together the stochastic models existing in rock mechanics to simulate the repartition of families of discontinuities. We can cite the orthogonal, Beacher, Veneziano, Voronoi, and Determinist statistical models among these models.

This work aims to study stability during the excavation phase of a road tunnel and analyze the effect of the distribution of discontinuities, in our case, the South Portal Zone (SPZ) of Tunnel 4 of Kherrata (Algeria). Firstly, the FEM is used with the equivalent medium Model. Then, a statistical model will combine the FEM and the DFN. Finally, the results obtained will be analyzed to determine the effect of taking the joints into account.

MATERIALS AND METHODS

Presentation of the Case Study: Tunnel 4 of Kherrata

Tunnel 4 is a set of four tunnels connecting the province of Sétif to Béjaïa, designed to accommodate heavy truck traffic. The tunnels pass throughout the Gorges of Kherrata, between Bordj Mira and Kherrata town. Tunnel 4 is situated between the chainages PK 6+175 and PK6 + 570 m of 395m in length. Figure 1 shows explicitly the parts of Tunnel 4: NPZ and SPZ and the Central area (Figure 3) [Boukarm et al.,2017]

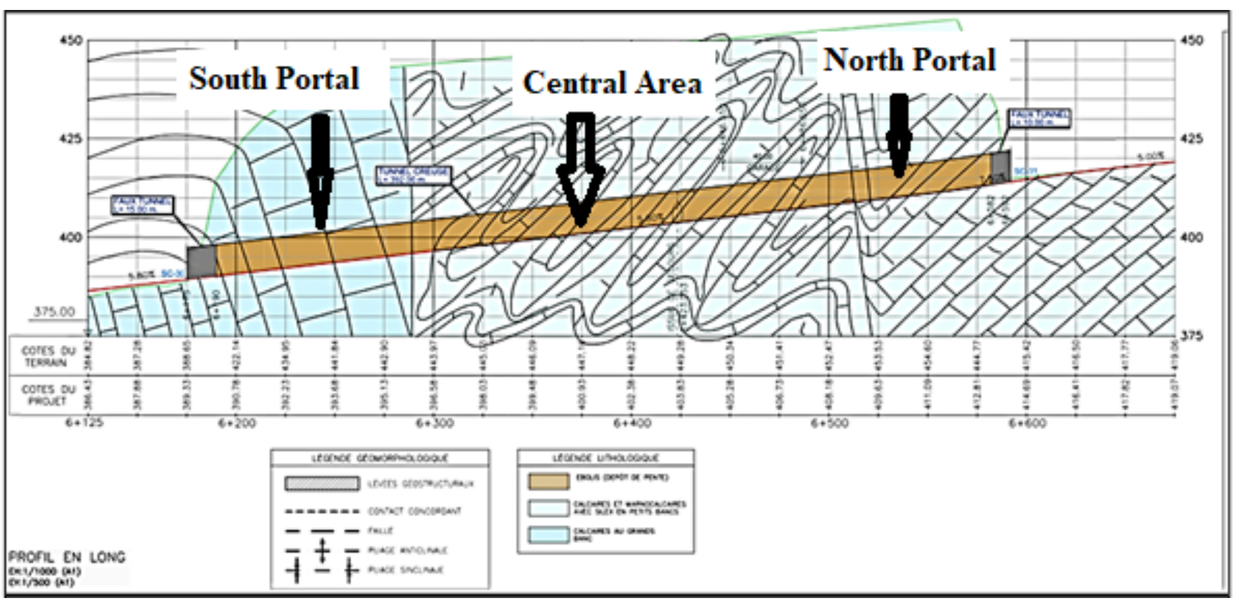

FIGURE 1. Profile of Tunnel 4 of Kherrata

Geological Settings

he Pera-Mediterranean Alpine orogeny of the Tertiary period, as described by Delga (1969), extends from southern Spain to the Calabro-Sicilian arc (Delga, 1969; Wildi, 1983). This orogenic event shaped the varied geological and structural zones found in northern Algeria, which forms part of the Maghrebid segment of the Alpine chain (see Figure 3). The Maghrebids, also known as the Tell within the Alpine domain, are characterized by a substantial sequence of predominantly marly sedimentary rocks. While the Maghrebids represent the external zones, the Kabylids—considered the internal zones—correspond to the flysch domain, which includes remnants of the Tethys Ocean and its former sedimentary cover.

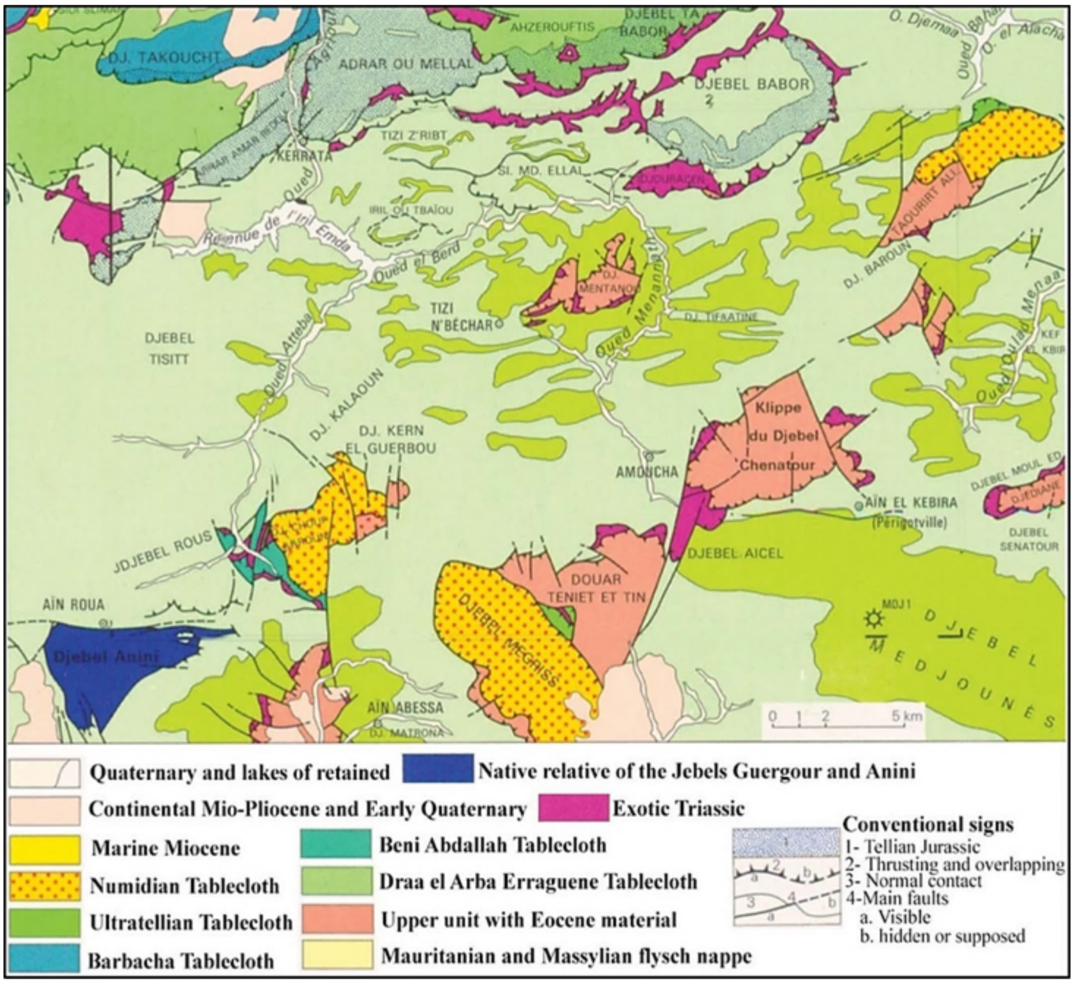

FIGURE 2. Structural geology map of the Tunnel project zone

The Kherrata Gorges form part of the Babor Mountain chain. The tunnel development is sited on Amar Rheddou Mountain. Petrologically, the project region is defined by Jurassic-period marly limestones. The rock mass is notably subject to brittle deformation, primarily due to the presence of numerous faults and joint systems. The observed joint density in the field has a major impact on the rock mass quality, leading to its division into two principal categories: jointed and moderately jointed rock. Additionally, weathering effects are present, linked to local wadis that are generally replenished by seasonal rainfall (Boukarm et al., 2023).

South Portal Area

The segment between Pk 6 + 500 and the end of Tunnel 4 is composed of grey limestone and marl limestone containing flint. The rock layers are almost vertical, dipping about 70° northward. These formations are characterized by fracturing, and the individual beds are over 2 meters thick.

Geomechanical Classification of the Study Area

Boukarm et al. (2017) conducted a geomechanical rock mass classification for the two areas, the North and South portal areas, using the most commonly employed classification methods: RMR and the Q-Barton. The table summarizes the results

TABLE 1. Values of geomechanical classification

Tunnel's 4 section	Length (m)	Value of RMR	Value of Q-Barton
NPZ	105	49	5.33
SPZ	70	33	1.06

The 02 classification methods yielded identical rock quality assessments for the examined zones: a fair rock and a poor rock, respectively, for NPZ and SPZ.

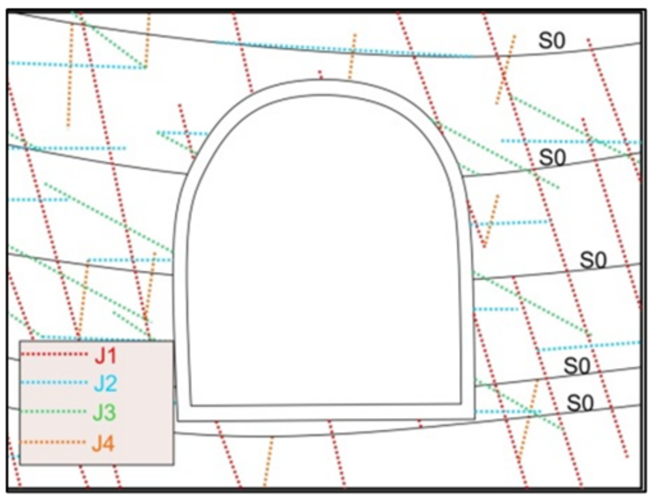

FIGURE 3. Geological mapping of joints at the South Portal of Tunnel 4 of Kherrata

METHODOLOGY

Two types of modeling are used to reach our objective. The first one utilized FEM as an equivalent medium model, and the second employed a model combining FEM and DFN Model (Crossed joint Model). Finally, we compare and analyze the results to evaluate the effect of joint spatial distribution on the simulation phase.

NUMERICAL SIMULATION ASSUMPTIONS

The numerical investigation employed Phase 2, a FEM analysis software package (Rocscience,2016). The analysis assumes a pseudo-continuous medium under plane strain conditions.

For Tunnel 4 (illustrated in Figure 6), the model incorporates six-node triangular elements. The mesh has a uniform configuration. The tunnel features a horseshoe-shaped geometry. Excavation boundaries were simulated in two sequential phases, replicating the heading-and-bench construction technique. Model boundaries were established at a distance equivalent to six tunnel diameters from the excavation periphery. Gravitational stresses were applied as the primary load condition, with material failure analyzed using the generalized Hoek-Brown criterion.

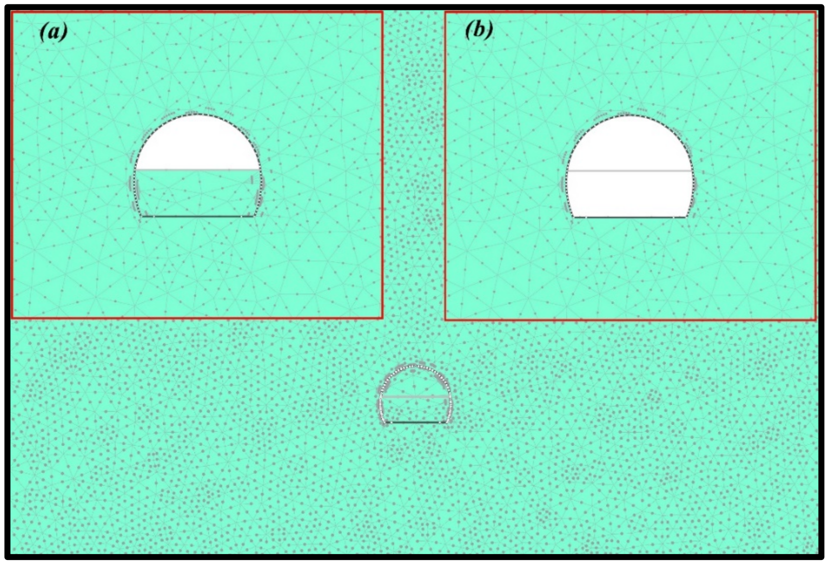

FIGURE 4. Geotechnical Model of the excavation phase by the Equivalent Model

The Crossed Joint Model represents the DFN model. There are sub-vertical layers and four sets of mapped joints. From the in situ visual field, the crossed-joint Model assumes that the sub-vertical layer and the crossing joint set (J1, as shown in the figure) predominate in the distribution. The bedding layer S0 and cross joints J1 have dip angle values of 70° and 30°, respectively. The Crossed joint Model follows the criterion of Mohr-Coulomb (MC) (table).

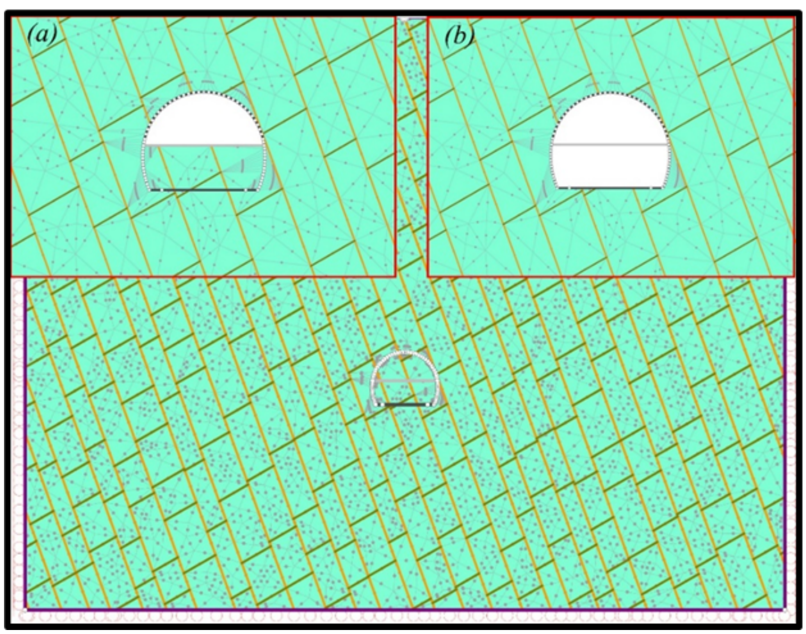

FIGURE 5. Geotechnical Model of the excavation phase by the Crossed Joint Model

Table 2 and 3 summarize the different physico-mechanical properties used during simulations. The figures represent the 02 numerical models used in numerical calculations.

TABLE 2. Different mechanical parameters of the SPZ

	Unit	Value
UCS	MPa	20
Young's modulus	MPa	3353.6
Poisson's ratio	-	0.3
Material constant	-	12
Unit weight	KN/m3	26
GSI	-	45

TABLE 3. Mechanical properties of the Croosed Joint Model with MC failure criterion

MC parameters	Unit	Value
Cohesion	MPa	0
Friction angle	°	30
Tensile strength	MPa	0

RESULTS AND DISCUSSION

The distances mentioned in the X-axis of the *graphs* and results correspond to specific structural zones of the tunnel cross-section:

- From 0 m to 9.4 m: This segment represents the *right wall* of the tunnel.
- From 9.41 m to 26 m: This interval corresponds to the *roof* (crown) of the tunnel.

From 26.33 m to 32 m: This range delineates the *left wall* of the tunnel.

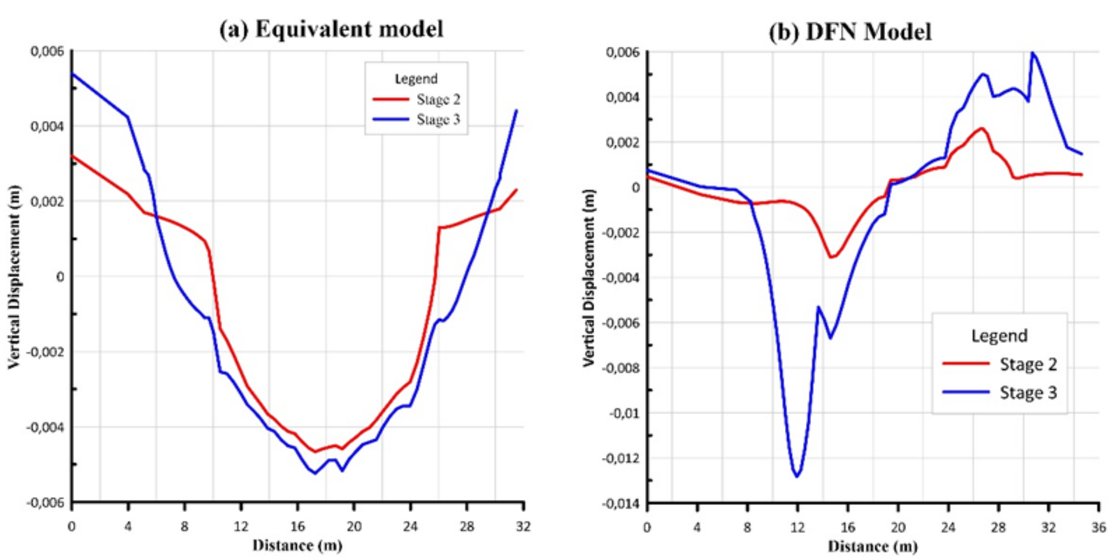

FIGURE 6. Evolution of vertical displacement during the excavation phases

The Equivalent Model shows vertical displacements from approximately 0.0055 m. The curves for Stage 2 and Stage 3 are relatively symmetric, with maxima near the roof (Figure 6). The DFN Model exhibits a broader range of vertical displacements, spanning approximately 0.005 m to 0.013 m. The curves are less smooth, with pronounced local variations, especially in Stage 3, which reaches a max value of nearly 0.013 m.

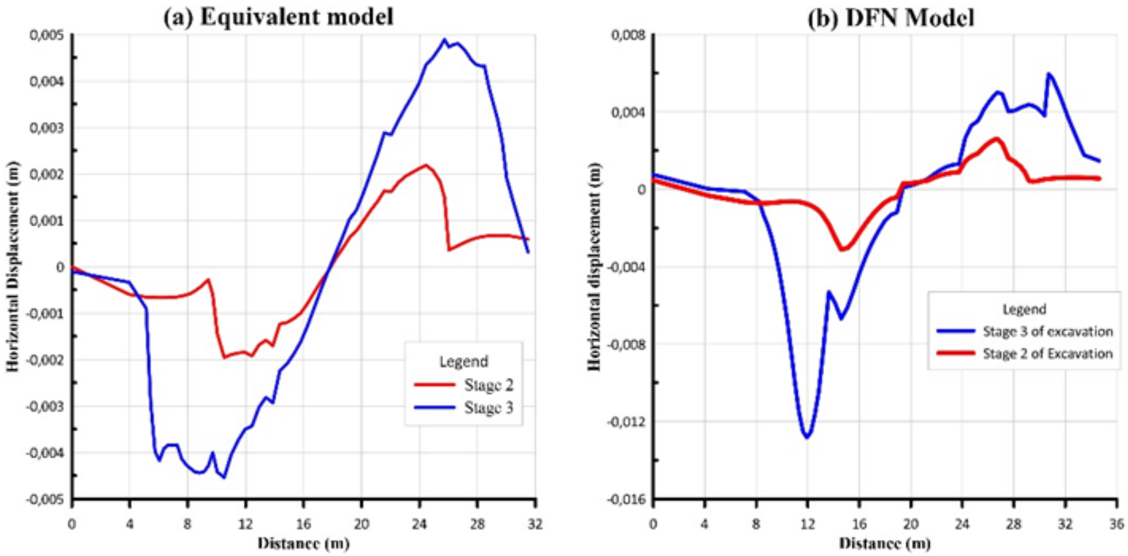

FIGURE 7. Evolution of horizontal displacement during the excavation phases

During stage 2, the Equivalent Model (Figure 7) shows horizontal displacements of approximately 0.0045 m. The curves are relatively smooth, with the largest displacements occurring along the length of the tunnel's crown. Stage 3 again shows greater displacement than Stage 2. In contrast, the DFN Model (Figure) exhibits horizontal displacements from about 0.007 m to 0.015 m, with more pronounced peaks, particularly in Stage 3. The curves are less regular, reflecting the influence of discrete fracture features.

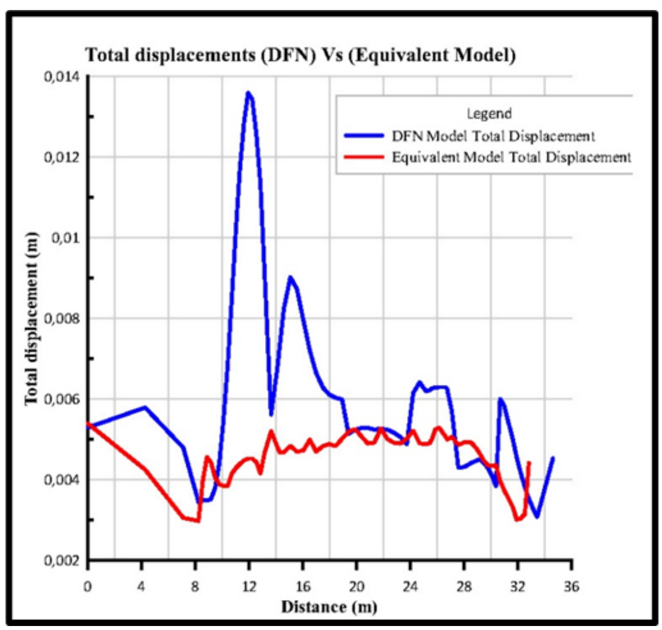

FIGURE 8. Evolution of total displacement from the DFN Model and the Equivalent Model

Overall, both the DFN and Equivalent Models display a similar trend in total displacement as distance increases (Figure 8). Notably, the DFN model tends to exhibit marginally higher total displacement values compared to the Equivalent Model across most of the distance range. This finding suggests that the DFN model, which explicitly represents fracture heterogeneity, predicts slightly greater deformation than the homogenized Equivalent Model.

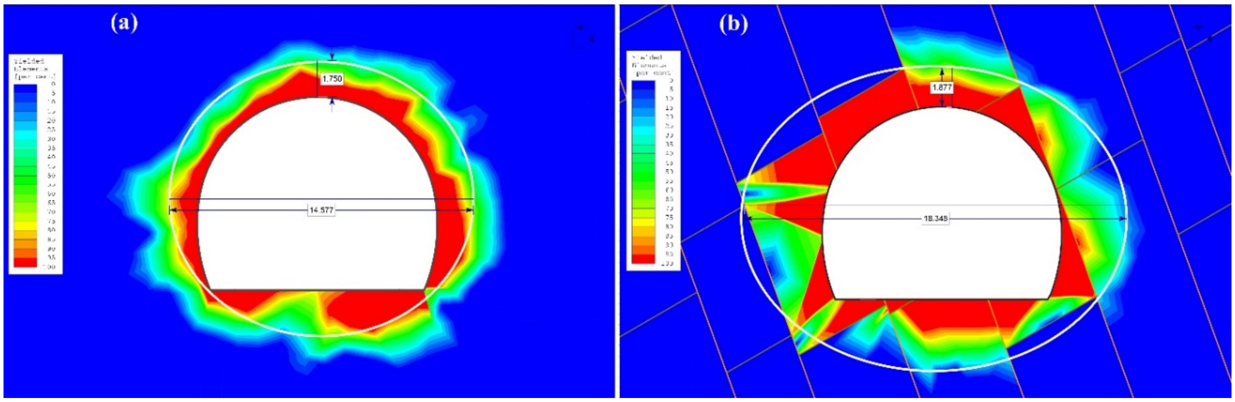

FIGURE 9. Plastic zone predicted by the Equivalent Model and the Crossed Joint Model

Figure 9 presents a comparative analysis of the plastic zones around the south portal, as predicted by two different modeling approaches. Subfigure (a) corresponds to the Equivalent Model, while subfigure (b) corresponds to the Discrete Fracture Network (DFN) Model. Both subfigures display the distribution of yielded elements, expressed as a percentage, using a color scale ranging from blue (low rate) to red (high percentage).

- In both models, the plastic zone—indicated by the concentration of red and orange colors—forms a continuous band around the periphery of the excavation, representing areas where the rock mass has yielded due to stress redistribution.
- In the Equivalent Model (a), the yielded zone is relatively uniform and symmetric, forming a nearly continuous halo around the opening. The maximum extent of the plastic zone is approximately 1.75 meters from the crown, and the horizontal span of the affected area is about 14.58 meters.
- In contrast, the DFN Model (b) shows a more irregular and anisotropic distribution of yielded elements. The plastic zone exhibits pronounced lobes and extensions, particularly along the directions aligned with the major fracture sets. The maximum extent of yielding is slightly greater, reaching approximately 1.88 meters from the crown, and the horizontal span of the affected area is wider at about 18.35 meters.

CONCLUSION

The equivalent continuous Model can provide some indications of the overall stability state. However, its continuity assumption limits a comprehensive understanding of the global behavior of the rock mass. Moreover, geometric parameters, specifically the orientation and spacing of joints, significantly influenced the simulation results of the deterministic Model relative to the Voronoi model. This finding underscores the critical role of joint distribution in stability analysis.

- The DFN Model also reveals localized zones of intense yielding, which correspond to the intersections and orientations of discrete fractures, resulting in a less uniform and more complex plastic zone geometry compared to the Equivalent Model.
- This figure demonstrates that the Equivalent Model predicts a more regular and symmetric plastic zone. At the same time, the DFN Model captures the influence of fracture heterogeneity, resulting in a more extensive and irregular yielded zone. The DFN Model's ability to represent localized zones of intense

plasticity highlights the importance of fracture networks in controlling the extent and shape of the plastic zone around underground excavations.

- The Equivalent Model can approximate the displacement behavior predicted by the DFN model; it may systematically underpredict the total displacement, especially in regions where fracture effects are significant. Therefore, for applications where precise displacement estimation is critical, the DFN model may offer superior accuracy due to its explicit representation of fracture networks.

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