

# A Case Study for 3D Verification of Convergence-Confinement Method for Multi-Staged Cavern Construction in Shear Zones

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**ABSTRACT:** The Convergence-Confinement Method (CCM) is a widely utilized approach for evaluating ground-structure interaction in deep underground excavations, such as caverns constructed using the Sequential Excavation Method (SEM), similar to tunnels. CCM assesses the radial displacement of the surrounding ground and the confining pressure mobilized by the support system. Ground Reaction Curves (GRCs), fundamental to this method, can be derived through analytical, semi-empirical, or numerical methods. Among the available methods, numerical approaches, such as the axisymmetric method that replicates stress-deformation behavior in radially symmetric excavation environments, provide accurate modeling of heterogeneous, anisotropic, or weak ground conditions above the tunnel. This paper presents a comparative study evaluating the deformation and loading behavior of support systems for an underground cavern designed for TBM assembly in Northern Istanbul. The cavern, measuring 27 m in height, 30 m in width, and 75 m in length, was constructed in 12 stages within low-strength rock and potential shear zones with soil-like behavior. The performance of CCM in these challenging conditions was assessed by comparing 2D analysis results with 3D simulations. The findings reveal that while the 2D models using CCM accurately estimate support forces comparable to those of 3D methods with differences around 20% at peak values, it is less reliable for predicting deformation behavior, as 2D displacements are approximately 70% higher than those obtained from 3D simulations. Thus, 3D methods are recommended for deformation-sensitive structures, while 2D CCM solution remains suitable for practical force estimations on support systems.

**KEYWORDS:** Cavern, Shear zone, NATM, SEM, Convergence-Confinement Method.

## 1 INTRODUCTION

Construction of underground caverns within geotechnically complex environments -particularly those affected by local low strength shear zones- demands rigorous modeling of ground-structure interaction to ensure both safety and structural reliability. Among the prevailing analytical frameworks, the Convergence-Confinement Method (CCM) has established itself as a reliable technique for evaluating these interactions, especially for tunnels constructed using Sequential Excavation Method (SEM) where phased construction influences stress redistribution and support behavior (Panet & Guenot, 1982; Carranza-Torres & Fairhurst, 2000).

While traditional two-dimensional (2D) numerical models have facilitated useful estimations of support pressures and load-bearing characteristics, their limitations in capturing true deformation patterns are notable when dealing with anisotropic, strain-softening, or soil-like/sheared rock masses (Alejano et al., 2003; Vlachopoulos & Diederichs, 2009). Meanwhile three-dimensional models, integrated with field data, offer enhanced resolution for deformation analysis, revealing insights that conventional 2D models often miss (Do & Dias, 2017; Bour et al., 2023).

This study aims to critically assess the reliability of the Convergence-Confinement Method (CCM) in predicting support forces and deformation behavior for multi-staged cavern excavation in shear-prone, low-strength rock, through an example of the study made for multi-staged cavern excavation in Northern Istanbul. By comparing 2D and 3D numerical simulations, the study identifies the practical boundaries of CCM's applicability, meanwhile highlighting that 2D solutions still remains a practical tool for support design despite limitations.

The cavern structure, measuring 30m width with 27m height and 75m length as given on Figure 1 and designated for Tunnel Boring Machine (TBM) assembly situated within low-strength rock masses exhibiting shear behavior. The primary concern during cavern construction is the presence of shear zones in the upper half of the excavation, as identified in the geological formation. The objective was to develop a support system capable of safely stabilizing the cavern within acceptable deformation limits, in case the expected shear zones

are encountered. Due to the cavern's large dimensions and geometrical constraints, including equipment size limitations, disallowing constructing excavation phases higher than 6.5 m- the excavation was divided into twelve sequential phases. This staging resulted in a gallery-like configuration with disproportionately wide top section, introducing additional challenges related to ground behavior, particularly given the excavation's deviation from circular geometry.

Another challenge was the necessity of producing a tunnel support solution for probable shear zones within the geological formation, in addition to the standard cavern design. During the design process, it was observed that the solution for shear zones yielded significant deformations and required intensive support system during the first three stages. Therefore, an additional study was conducted to ensure that the 2D analysis did not produce unnecessary overdesign while still providing a safe solution. As the cavern was successfully completed in 2024, without encountering any shear zone, the results could not be compared with field data; however, the comparative study of 2D and 3D results is considered to provide important outcomes for the application of CCM methods in simulating realistic tunnel behavior for multi-stage, non-circular, gallery-like tunnel excavations.

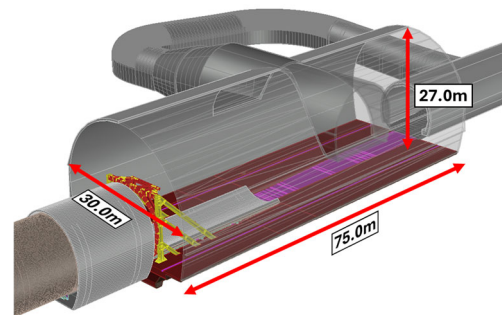


Figure 1. Cavern tunnel geometry.

## 2 CONVERGENCE-CONFINEMENT METHOD (CCM)

The Convergence-Confinement Method (CCM) is a widely adopted analytical, semi-empirical and numerical framework for evaluating ground-support interaction in underground

excavations, particularly tunnels and caverns constructed using the Sequential Excavation Method (SEM). It provides a simplified yet powerful approach to assess the deformation behavior of the surrounding ground and the mobilization of support pressure as excavation progresses.

### 2.1 Component of CCM

CCM is built upon three interrelated curves that describe the mechanical response of the ground and support system of tunnels. These include Ground Reaction Curve (GRC), Support Characteristic Curve (SCC) and Longitudinal Displacement Profile (LDP).

GRC, SCC and LDP form the foundation of conventional tunnel design. These methods provide a basis for planning excavation support systems—including the maximum unsupported excavation step, primary shotcrete lining, reinforcement type, and additional support elements—based on input parameters such as excavation geometry, overburden depth, and geological conditions.

#### 2.1.1 Ground Reaction Curve (GRC)

Ground Reaction Curve (GRC) represents the relationship between internal pressure acting on the tunnel boundary and resulting radial displacement (convergence) of the surrounding ground due to excavation. In the case of an unsupported tunnel, the internal pressure acting on the tunnel boundary ( $p_i$ ) matches the in-situ ground stress ( $p_o$ ) at a point located ahead of the excavation face. This pressure gradually reduces to zero moving backward from that point, opposite to the direction in which the tunnel is being advanced, as illustrated in Figure 2.

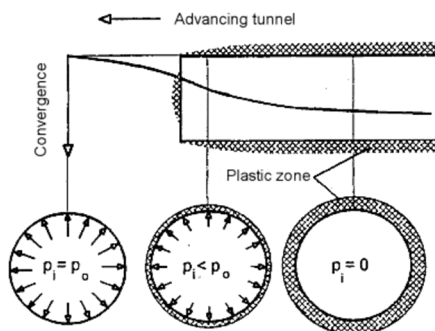


Figure 2. Assumed support pressure  $p_i$  at different positions relative to the advancing tunnel face (Hoek, 1999).

#### 2.1.2 Support Characteristic Curve (SCC)

Support Characteristic Curve (SCC) is a load–displacement ( $\sigma_r$ – $u_r$ ) curve describes how the support system reacts to increasing deformation, based on its stiffness and installation timing. It is a critical component of the CCM, used to evaluate how support elements resist ground deformation and interact with the surrounding rock mass.

The intersection of the SCC with the Ground Reaction Curve (GRC) defines the equilibrium point of ground–support interaction as shown in Figure 3. The important practical aspect is that support should be installed soon enough to prevent loosening of the rock mass but late enough to prevent overloading through plastic yield.

#### 2.1.3 Longitudinal Displacement Profile (LDP)

Longitudinal displacement profile (LDP) captures the evolution of radial displacement along the tunnel axis, both ahead of and behind the excavation face. It is essential for determining the optimal location and timing of support installation (Panet, 1995).

The graph as shown below Figure 3 longitudinally represents the radial displacement of the ground around the

tunnel during excavation. Determining the LDP during tunnel excavations according to the rock mass conditions is essential for thorough tunnel behavior analyses (Ha et al., 2021).

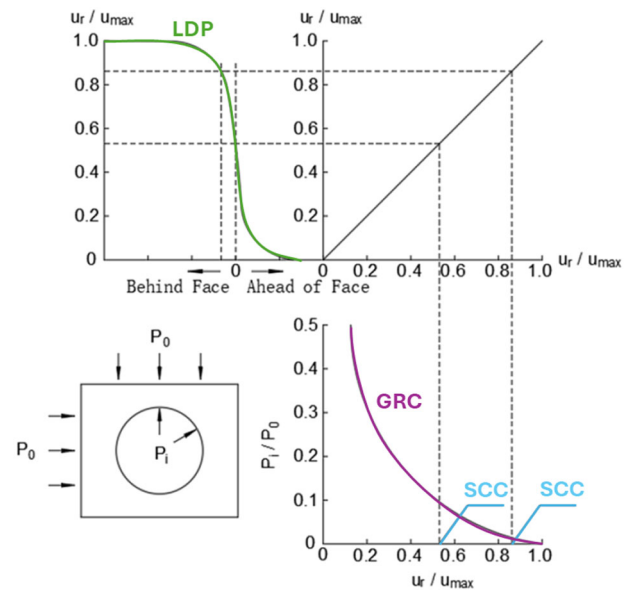


Figure 3. GRC, SRC and LDP relationship (Zhong et al., 2023).

### 2.2 Implementation Practices of CCM

#### 2.2.1 Analytical implementation of CCM

Analytical methods form the foundation of CCM and are based on closed-form solutions derived from elasticity and plasticity theory. Analytical approach generally assumes axisymmetric geometry and homogeneous ground conditions. Kirsch (1898) established the baseline by analyzing elastic stress distributions around circular excavations. Fenner (1938) extended this framework through the incorporation of plastic deformation zones. The GRC was later articulated by Panet and Guenot (1982), facilitating the quantitative linkage between support pressures and tunnel wall convergence. Subsequently, Carranza-Torres and Fairhurst (2000) integrated Hoek-Brown failure criterion, thereby enhancing the predictive capacity of CCM models under non-linear rock mass conditions.

However, despite its analytical utility, the approach is inherently limited in its ability to capture complex geomechanical behaviors such as anisotropy, and non-standard stress distributions, and generally assumes idealized tunnel geometries, which may diverge from field realities.

#### 2.2.2 Semi-empirical implementation of CCM

Semi-empirical method represents an approach that blends analytical principles with empirical field observations and simplified numerical outputs. These methods commonly employ curve-fitting techniques to generate LDPs. They explicitly account for support installation timing, face advance effects, and other factors influencing excavation-induced deformations. Panet (1995) introduced empirical LDPs for unsupported tunnels. This foundation was extended by Vlachopoulos and Diederichs (2009), who enhanced the formulation using plastic radius theory and displacement data. Further refinement by Oke et al. (2018) incorporated support proximity and stiffness effects, significantly improving the model's responsiveness to support interaction.

Semi-empirical CCM techniques are particularly valuable for preliminary design, estimating support loads, and identifying deformation zones, as well as for validating analytical models against site-specific field measurements.

### 2.2.3 Numerical implementation of CCM

Numerical modeling enhances the applicability of CCM by allowing simulation of complex ground conditions and excavation sequences. Method can be implemented by using:

- 2D axisymmetric models can be applied to circular tunnels when the stress distribution between the crown and the invert is sufficiently similar, allowing the assumption of radial symmetry. These models simulate stress redistribution and deformation using simplified geometry and are computationally efficient.
- 2D plane strain models which can be used for elongated excavations where radial symmetry is not applicable. These models can incorporate staged excavation and support installation but may oversimplify three-dimensional effects.
- 3D finite element models which can provide a more realistic representation of excavation geometry, geological heterogeneity, and sequential construction. They allow for detailed analysis of stress paths, plastic zones, and support interaction, especially in anisotropic or shear-prone ground.

CCM is typically implemented in numerical models by simulating excavation stages, monitoring ground convergence, applying support elements at predefined distances from the face, extracting GRC and SCC from simulation results to evaluate interaction and comparing LDP profiles with field measurements to calibrate model behavior.

## 3 METHODOLOGY

### 3.1 Geological and Geotechnical Conditions

The location for Cavern tunnel which is in northern Istanbul along a geological setting mainly consists of Garipçe Formation (KsgA). This formation contains complex lithology of cataclastic lava and pyroclastic rocks (Tüysüz & Balkaya, 2024). Site investigations of the area show that the geomechanical quality of the formation gradually improves with increasing depth. Meanwhile, the alignment is frequently disrupted by dykes, faults, and shear zones characterized by very low strength and soil-like mechanical behavior, which are also infamous for being the cause of unexpected behavior and partial collapses in tunnels constructed in the region with NATM/SEM method (Sahin, 2010).

The cavern location is selected after a comprehensive series of geological studies to minimize the probability of encountering shear zones at excavation depth, meanwhile several constraints like construction schedule, interaction with urbanized environments etc. influenced the final location.

The top heading of the cavern was planned to be constructed at a depth approximately 60 m below ground level, within andesite formations exhibiting variable uniaxial compressive strength, averaging around 15 MPa, and GSI values between 35 and 50. In this regard, while it was expected that cavern excavation will take place mostly in low to medium strength rocks classified as B2 and B3 according to ÖNORM B 2203, the possibility of encountering shear zones required an additional study to select the support system and additional measures to avoid any incident that risks construction safety.

The idealized geological profile, developed to assess the potential intersection with shear zones within B3-class rock, along with representative geotechnical parameters for the identified geological units, is presented in Table 1. The equivalent Mohr-Coulomb ( $c'$  and  $\phi$ ) parameters presented in Table 1, which were used in the numerical analyses, were derived from the field-observation-based Hoek-Brown (HB) criterion. This conversion was performed using the Rocscience

RocData software, utilizing rock mass parameters (GSI,  $m_i$ , D). RocData employs the analytical method, optimized for the expected stress range around the tunnel, developed by Hoek, Carranza-Torres, and Corkum (2002). The rock mass deformation modulus ( $E_m$ ) values were estimated using the empirical relationships proposed by Hoek and Diederichs (2006). This methodology ensures that the parameters presented in Table 1 reliably represent the rock mass behavior under the relevant stress conditions. Based on this profile, it is anticipated that the top heading stages will be entirely situated within the shear zone, with approximately 12 m of this weak layer overlying the crown of the excavation.

Table 1. Profile and Equivalent Mohr-Coulomb Parameters at Cavern Location

Unit	App. Depth	$\gamma$ (kN/m <sup>3</sup> )	$c'$ (MPa)	$\phi'$ (°)	$E_m$ (MPa)
W6-KsgAP	0* - 8.5	20	0.035	40	83
Ksg-1a	8.5 - 13	23	0.080	59	790
Shear Z-1	13 - 26	20	0.037	41	73
Ksg-1b	26 - 36	23	0.146	52	790
Shear Z-2	36 - 44	20	0.062	35	250
Ksg-1c	44 - 48	23	0.191	49	790
Shear Z-3	48 - 68	20	0.081	32	250
Ksg-1d	68 - ...	23	0.326	43	790

\*: 0 m corresponds to the ground surface at +122 m above sea level.

Dimensions are given in meters (m), deformations and thicknesses in millimeters (mm), and strengths in megapascals (MPa); unit consistency was checked throughout the text.

### 3.2 Support System

Multi-stage excavation strategy of Sequential Excavation Method (SEM) was used due to the cavern's large cross section, equipment capabilities and the challenging geotechnical conditions of the site. To maintain cavern stability and control ground movement during construction, the excavation was divided into 12 phases, as illustrated in Figure 4. Strategically, the top left (T1) and right (T2) side drifts are excavated first, to investigate ground conditions and take action for further stages if necessary. Following the support of these drifts, the central mass between them (T3) is excavated to safely establish the final profile of the crown, as shown in Figure 5. Once the top heading arch is complete, the excavation proceeds downwards with the removal of the upper bench sections. As this paper focuses on the comparison of the behavior of heading arch which is likely to be excavated in sheared rock zones, the details of the bottom excavation levels in rock will not be included in this paper.

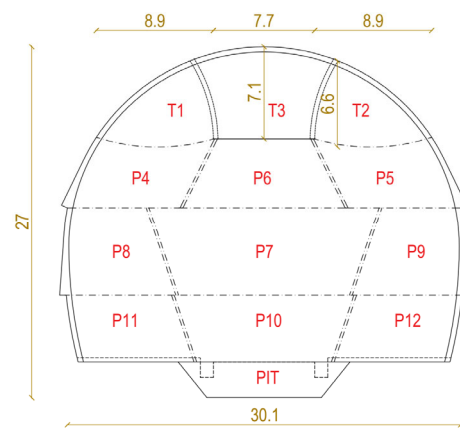


Figure 4. Schematic of multi-stage excavation sequence.

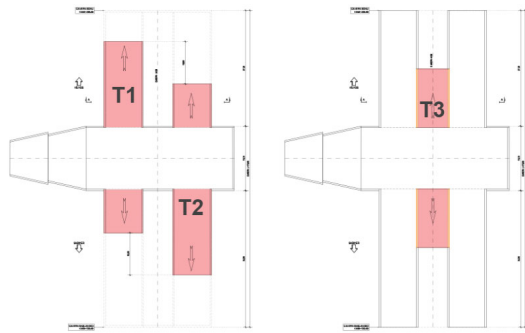


Figure 5. Stages of Excavation Advancement at heading arch.

To guarantee the stability of the excavation under various geological circumstances, the cavern tunnel's support system was carefully planned and put into place in numerical models. The primary lining to be applied for shear zones, included HEB260 steel profiles with 0.5 m spacing placed at every 1.0 m advancement (excavation progress length), combined with a 400 mm thick layer of reinforced shotcrete and wire mesh, supplemented by an extensive 12.0 m self-drilling rock bolting scheme. The support scheme is considered to provide an improved arch above the tunnel crown, which is considered in models as a composite improvement zone. At the tunnel face, where the tunnel advances in shear zones, 9.0 m long self-drilling rock bolts were planned along with a 100 mm reinforced shotcrete with wire meshes to ensure face stability. A lighter rock bolting scheme was adopted for the internal walls of the T1 and T2 drifts as shown in Figure 6. Based on multiple analysis cases, these internal walls were planned to be removed during the T3 excavation phase, as shown in Figure 7. Their structural contribution was found to be limited after the central drift excavation, and sustaining the increased axial forces would have required a significantly heavier support system, especially as the internal walls began to function like columns following the middle drift removal.

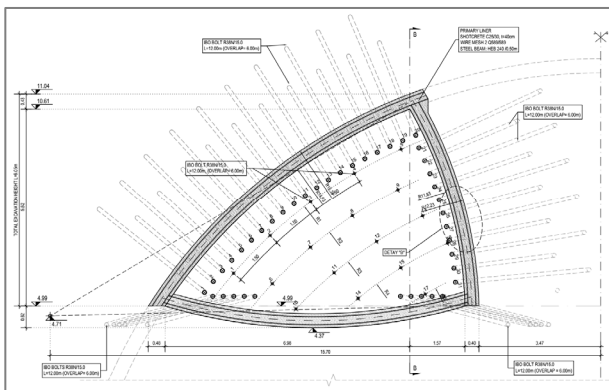


Figure 6. Support scheme of T1 & T2 drifts.

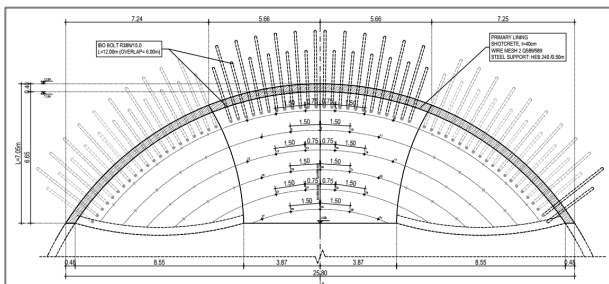


Figure 7. Support scheme of T3 drift.

### 3.3 Numerical Model Setup – Plaxis 2D

In Plaxis 2D, a plane strain model was made to show a typical cross-section of the cavern tunnel and the different stages of excavation. This model is mostly used to investigate into how the rock mass and support system interacted and behaved in the transverse direction. To reduce the effects of the boundaries on the tunnel's behavior, the model boundaries were selected according to the cavern dimensions resulting in a model size of 120 m in width, 125 m in height; and checked against interaction. The vertical sides were restrained against horizontal displacement while remaining free vertically, the bottom boundary was fixed in both directions, and the ground surface was left free to deform under excavation-induced stresses. The mesh was made up of triangular elements with 15 nodes. A much finer mesh was used around the tunnel excavation and support elements to get precise readings of stress concentrations and deformations. The Mohr-Coulomb soil model and equivalent Mohr-Coulomb parameters, calculated according to the Generalized Hoek-Brown Criterion for the relevant depth as explained in Chapter 3.1, were used for the rock units. Meanwhile shear zones were modelled using “Hardening Soil (Isotropic Hardening) Model”.

The Convergence-Confinement Method (CCM) was added to take into account the three-dimensional effects that come with excavating tunnels, and numerical implementation of CCM was considered in order to overcome the shortcomings of analytical and semi-empirical methods regarding sheared and soil-like behaving rocks.

The 3D face-proximity effect (LDP) was mapped in the 2D plane strain model through a two-stage stress release process: a relaxation factor was applied to initially release a portion of the stress down to the  $\pi/p_0$  ratio, which defines the optimal support installation timing and the starting point of the Support Characteristic Curve (SCC); the remaining stress was then released as the SCC is mobilized to find final equilibrium on the Ground Reaction Curve (GRC). As a result, following deconfinement  $(1 - \beta)$  values were determined for the first 3 stages to be compared.

Table 2. Calculated Deconfinement  $(1 - \beta)$  Values

Stage	At Tunnel Face	At 1 m distance
T1	0.26	0.69
T2	0.26	0.69
T3	0.31	0.75

The rigidity and geometrical dimensions of the liners and tunnel reinforcements were explicitly incorporated into both the 2D and 3D models, with the composite lining represented by a 400 mm reinforced shotcrete layer combined with HEB260 steel profiles spaced at 0.5 m intervals. In addition, the time-dependent aging effects of shotcrete were considered to realistically capture stiffness development and long-term support behavior. The axial rigidity (EA) was defined as  $1.07 \times 10^7$  kN/m at 2 days and  $1.50 \times 10^7$  kN/m at 28 days, while the flexural rigidity (EI) was set to  $1.44 \times 10^5$  kNm<sup>2</sup>/m at 2 days and  $2.05 \times 10^5$  kNm<sup>2</sup>/m at 28 days according to composite strength evaluation approach by Carranza-Torres (2004). The improved tunnel face was represented by composite ground parameters considering the area ratio of bolts and ground, ensuring that the models reflected both structural rigidity and enhanced ground-support interaction.

### 3.4 Numerical Model Setup – Plaxis 3D

Given that the cavern construction was anticipated to intersect shear zones extending over a length of approximately 10

meters, at the end of the cavern area, a detailed three-dimensional numerical model was developed to accurately simulate ground behavior under such conditions. To minimize boundary effects and ensure computational stability, the model domain was defined as 120 m in width, 125 m in height, and 50 m in length, with the excavation itself represented over a 10 m section -corresponding to the most critical zone affected by shearing, as shown in Figure 8. The model domain was checked to verify that it has extended well beyond the cavern dimensions to minimize boundary effects. Mimicking the same boundary conditions with 2D model, the bottom boundary was fixed in all directions, while the vertical sides were restrained laterally but allowed vertical movement. The ground surface was left free to deform under excavation-induced stresses. Same material models were used as in PLAXIS 2D.

Sequential excavation was simulated in accordance with the planned construction stages, with each phase precisely replicated to reflect actual excavation geometry and timing, consisting of 93 calculation phases only to model first 3 phases of heading arch. While this increased the computation time dramatically, this approach enabled realistic representation of face advance, ground relaxation, and support mobilization, thereby capturing three-dimensional stress redistribution and deformation mechanisms associated with staged tunneling through weak, sheared rock formations.

It must be noted that excluding the lower stages occurring in sound rock is considered a limitation of this study, as it may influence the total cumulative deformations and the absolute peak values of certain internal forces in a complete model. On the other hand, the results obtained for the first three stages within the shear zone serve the intended purpose of comparing the 2D CCM methodology with 3D results, for non-circular gallery-like geometry, under such sheared rock conditions.

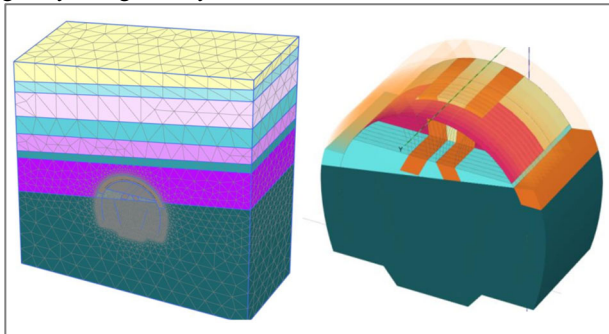


Figure 8. 3D FEM Model in PLAXIS 3D  
a. Overall model b. Excavation Phases.

#### 4 COMPARISON OF RESULTS

According to the results, one of the key differences between the 2D and 3D models was the estimation of liner deformations. In the 3D model, deformations increased progressively with excavation advancement, reaching a peak of approximately 19 mm - a relatively modest value for sheared zone, attributed to the presence of an extensive support system. In contrast, the 2D model, which assumes excavation along an infinitely long section, estimated a maximum liner deformation of 69 mm. 3D analysis predicted a value approximately 70% lower than predicted by the 2D analysis. This difference can be mechanistically explained by finite-length and face-effect stiffness in the 3D model, where the advancing tunnel face provides additional confinement and redistributes stresses, thereby reducing crown displacement. Furthermore, the staged installation sequence in 3D allows progressive mobilization of ground support and stress arching, limiting deformations compared to the plane-strain assumption of 2D.

With regard to liner forces, localized peaks were observed at connection points in the 3D analysis. These peaks were considered numerical stress concentration artifacts rather than representing the global structural capacity or average section demand. These peaks were filtered from the final evaluation due to their highly localized nature. The criterion for filtering was based on discarding localized values (spanning less than 0.5 m) that exceeded the adjacent average liner demand by more than 40%. Overall, forces acting on the liner were relatively uniform along the longitudinal axis, justifying the use of a representative section extracted from the midpoint of the cavern lining. The radial force distribution demonstrated reasonable agreement with 2D model results. A summary of the peak liner forces is provided in Figure 9.

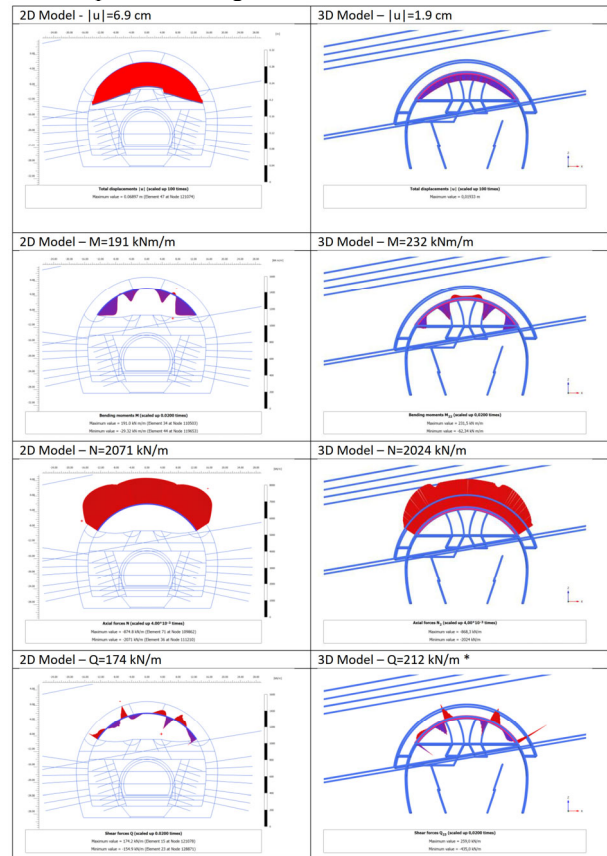


Figure 9. Comparison of 2D and 3D Results.

The 3D analysis revealed an approximately 20% increase in peak bending moments and peak shear forces compared to the 2D model, while peak axial forces remained largely consistent across both modeling approaches. These increased values, when assessed via interaction charts, remained within acceptable design thresholds. The radial distribution of axial and moment forces was evaluated using interaction charts based on composite section behavior as well as considering load distribution between the shotcrete lining and the HEB260 steel profiles, in accordance with the principles outlined by Carranza-Torres (2000). The primary lining (HEB260 + 400 mm shotcrete) was modeled assuming full composite action (i.e., neglecting potential bond-slip), ensuring the full stiffness sharing capacity of the components was utilized. The M–N interaction curves were obtained via an equivalent section analysis approach, based on ultimate limit state (ULS) principles. The comparison of moment-normal force (MN) distributions is presented in Figure 10.

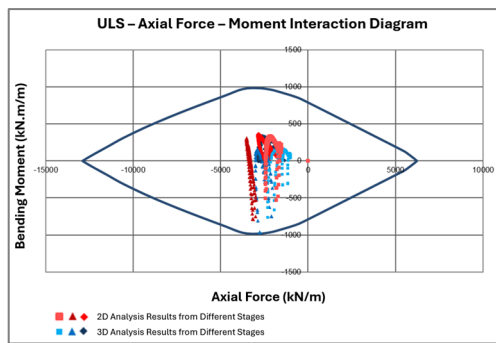


Figure 10. Interaction Chart and distribution of M-N values from 2D and 3D Analysis Results

## 5 CONCLUSIONS

This study highlights the comparative behavior of tunnel lining responses for large and wide cavern geometries in weak rock conditions, as predicted by 2D and 3D numerical models, emphasizing the strengths and limitations of each approach. According to the results:

- 3D analysis, which captures excavation advance and finite-length effects, predicts substantially lower liner deformations approximately 70% lower than the conventional 2D infinite-length assumption.
- Despite the localized, highly confined peaks at connection points in 3D, forces along the longitudinal axis are repetitive, and a midspan section provides results consistent with 2D; however, 3D yields approximately 20% higher peak bending moments and shear while peak axial forces remain comparable.
- Interaction-chart evaluations of the composite lining (shotcrete + HEB260), with load sharing guided by Carranza-Torres (2000), confirm that all demand levels are within allowable limits, including 2D analysis.

Practically, 2D analysis remains a conservative tool for deformation envelopes, whereas 3D is essential to capture realistic convergence, construction staging, and local effects, supporting efficient yet safe support design.

On the other hand, results indicate that 2D analysis, combined with the Convergence-Confinement Method (CCM), remains a practical and efficient tool for predicting support behavior and verifying tunnel design even for gallery-like non-circular cavern structures - especially given that both models confirm acceptable interaction margins. Considering that 3D analysis requires significantly greater modeling effort and computational resources to produce relatively more optimized results, 2D analysis proves to be a more accessible and pragmatic alternative in most cases.

Nonetheless, where deformation control is critical or the 2D results approach design thresholds, transitioning to 3D modeling becomes justified.

Future work should couple these models with field monitoring for calibration and explore parameter sensitivity (support stiffness, excavation length, and connection detailing) to further optimize performance.

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