

**OPTIMIZATION OF SEGMENTAL TUNNEL LININGS: A
PARAMETRIC STUDY ON THE EFFECTS OF GSI AND
OVERBURDEN DEPTH**

**SEGMENT KAPLAMALARININ OPTİMİZASYONU: GSI
VE ÖRTÜ KALINLIĞININ ETKİLERİ ÜZERİNE
PARAMETRİK BİR ÇALIŞMA**

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ABSTRACT

OPTIMIZATION OF SEGMENTAL TUNNEL LININGS: A PARAMETRIC STUDY ON THE EFFECTS OF GSI AND OVERBURDEN DEPTH

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In mechanized tunneling using Tunnel Boring Machines (TBMs), accommodating severe geostatic loading typically requires increasing the thickness of precast concrete segmental linings, which introduces critical logistical constraints, elevates excavation volumes, and increases overall material consumption. To address these engineering challenges, this thesis presents a comprehensive numerical investigation into an alternative support strategy that couples thin precast concrete segments with internal, high-stiffness structural steel rings installed in an alternating sequence. Three-dimensional finite element simulations were conducted using PLAXIS 3D to capture the multi-axial ground-structure interaction and staged excavation dynamics of mechanized shield tunneling, employing the elastoplastic Hoek-Brown failure criterion for the surrounding rock mass. The system was evaluated in two main directions: (a) variation of the Geological Strength Index (GSI) with a constant depth and (b) variation of the operating overburden depth with uniform ground conditions, using a complete parametric framework. The structural response was systematically analyzed by means of key performance indicators such as total displacements, hoop thrust forces, in-plane shear forces, in-plane bending moments, as well as secondary longitudinal and out-of-plane internal forces. The numerical results demonstrate that the alternating

stiffness configuration activates an active load-redistribution mechanism along the tunnel drive; the structural steel rings of high stiffness receive disproportionately higher axial forces and flexural demands, thus shielding the neighboring thin concrete segments from localized stress concentrations and eccentric loading. The longitudinal force profiles show a typical periodic variation with demand maxima at the locations of the steel ring and a significant relief in the zones of the intermediate concrete segment. Internal force magnitude is strongly dependent on rock mass competence and a better ground quality triggered by mobilization of an effective natural ground arch leads to a sharp reduction of structural bending demands. Rather, an increase in overburden depth will raise the cross-sectional thrust and sidewall bending actions. However, the structural utilization levels of all components were observed to remain within the allowable limit state thresholds under the investigated conditions. To summarize, the study demonstrates the potential applicability of internal steel rings to significantly reduce the lining thickness compared to conventional solutions, and provides a structurally stable, logistically feasible and material-efficient alternative for deep mechanized tunneling in challenging geological conditions.

Keywords: Mechanized Tunneling, Segmental Linings, Steel Ring Reinforcement, Structural Optimization, Numerical Modeling, Hoek-Brown Criterion.

ÖZET

SEGMENT KAPLAMALARININ OPTİMİZASYONU: GSI VE ÖRTÜ KALINLIĞININ ETKİLERİ ÜZERİNE PARAMETRİK BİR ÇALIŞMA

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Tünel Delme Makineleri (TBM'ler) kullanılarak gerçekleştirilen mekanize tünel açma çalışmalarında, şiddetli jeostatik yüklerin karşılanması genellikle ön dökümlü beton segment kaplamaların kalınlığının artırılmasını gerektirir. Bu durum, ciddi lojistik kısıtlamalara yol açmakta, kazı hacimlerini artırmakta ve genel malzeme tüketimini yükseltmektedir. Bu mühendislik zorluklarını aşmak amacıyla, bu tez, ince prefabrik beton segmentleri ile iç kısımda belli aralıklarla yerleştirilen yüksek rijitlikli yapısal çelik halkaları birleştiren alternatif bir destek stratejisine yönelik kapsamlı bir sayısal inceleme sunmaktadır. PLAXIS 3D kullanılarak, çok eksenli zemin-yapı etkileşimini ve mekanize kalkanlı tünel açma işleminin aşamalı kazı dinamiklerini görmek üzere üç boyutlu sonlu eleman simülasyonları gerçekleştirilmiş ve çevreleyen kaya kütlesi için elastoplastik Hoek-Brown yenilme kriteri kullanılmıştır. Sistem, eksiksiz bir parametrik çerçeve kullanılarak iki ana yönde değerlendirilmiştir: (a) sabit derinlikte Jeolojik Dayanım Endeksi'nin (GSI) değişimi ve (b) sabit zemin koşullarında örtü kalınlığının değişimi. Yapısal tepki, toplam yer değiştirmeler, dairesel itme kuvvetleri, düzlem içi kesme kuvvetleri, düzlem içi eğilme momentleri ile ikincil boyuna ve düzlem dışı iç kuvvetler gibi temel performans göstergeleri aracılığıyla sistematik olarak analiz edilmiştir. Sayısal

sonular, deęiřken sertlik konfigürasyonunun tünel açma hattı boyunca aktif bir yük dağıtım mekanizmasını harekete geçirdiğini göstermektedir. Yüksek sertliğe sahip yapısal elik halkalar, orantısız şekilde daha yüksek eksenel kuvvetler ve eğilme yükleri olarak, komřu ince beton segmentlerini yerel gerilme yoğunlařmalarından ve eksantrik yüklemelerden korumaktadır. Boyuna kuvvet profilleri, elik halkanın bulunduęu yerlerde yükün maksimum deęerine ulařtığı ve ara beton segment bölgelerinde önemli bir azalma görüldüğü tipik bir periyodik deęişim göstermektedir. İç kuvvetlerin büyüklüğü, kaya kütlesinin dayanıklılığına büyük ölçüde baęlıdır; etkili bir doęal zemin kemerinin harekete geçirilmesiyle saęlanan daha iyi zemin kalitesi, yapısal eğilme yüklerinde keskin bir azalmaya yol açmaktadır. Öte yandan, örtü tabakasının derinliğindeki bir artış, kesitsel itme kuvvetini ve yan duvar eğilme etkilerini artırmaktadır. Ancak tüm bileřenlerin yapısal kullanım seviyeleri, izin verilen sınır durum eřiklerinin altında kalmaktadır. Özetle, bu alıřma, geleneksel özömlere kıyasla kaplama kalınlığını azaltmada iç elik halkaların uygulanabilir potansiyeline işareet etmekte ve zorlu jeolojik kořullarda derin mekanize tünel açma alıřmaları için yapısal açıdan dengeli, lojistik olarak uygulanabilir ve malzeme tasarrufu saęlayan bir alternatif sunmaktadır.

Anahtar Kelimeler: Mekanize Tünelcilik, Segment Kaplamalar, elik Halka Donatı, Yapısal Optimizasyon, Sayısal Modelleme, Hoek-Brown Kriteri.

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SYMBOLS AND ABBREVIATIONS

Symbols

γ	Unit weight
ν	Poisson's ratio
ν_{12}	In-plane Poisson's ratio
σ_{ci}	Intact uniaxial compressive strength
σ_{cm}	Rock mass compressive strength
σ_h	Horizontal stress
σ_v	Vertical geostatic stress
d	Lining structural thickness
D	Disturbance factor (Hoek-Brown)
D	Tunnel outer diameter
D_i	Tunnel internal diameter
E_1, E_2	Young's modulus in principal directions
E'_{rm}	Rock mass deformation modulus
G_{12}, G_{13}, G_{23}	Shear moduli in principal planes
H	Overburden depth
k	Subgrade reaction modulus
K_0	At-rest earth pressure coefficient
m_i	Intact rock constant (Hoek-Brown)
M_{11}	In-plane bending moment
M_{12}	In-plane torsional moment
M_{22}	Out-of-plane longitudinal bending moment
N_1	Longitudinal axial force
N_2	Circumferential hoop axial force

Q_{12}	In-plane transverse shear force
Q_{13}	Out-of-plane longitudinal shear force
Q_{23}	Out-of-plane circumferential shear force
t	Lining thickness
u	Total deformation / displacement
$ u $	Resultant total displacement magnitude
X, Y, Z	Cartesian coordinate axes

Abbreviations

2D	Two-Dimensional
3D	Three-Dimensional
CAD	Computer-Aided Design
CSL	Concrete Segmental Lining
EPBM	Earth Pressure Balance Machine
FEA	Finite Element Analysis
FEM	Finite Element Method
GSI	Geological Strength Index
KPI	Key Performance Indicator
LCA	Life-Cycle Assessment
NATM	New Austrian Tunneling Method
SFRC	Steel Fiber Reinforced Concrete
TBM	Tunnel Boring Machine
ULS	Ultimate Limit State

1. INTRODUCTION

1.1. Background and Motivation

It is observed that the escalating demands of global urbanization and the continuous necessity for robust infrastructure have made the construction of underground facilities a vital aspect of modern engineering (Bäppler, 2016; Bilgin & Acun, 2024). In this context, mechanized tunnelling, predominantly carried out by Tunnel Boring Machines (TBMs), has become a widely adopted and highly efficient method for developing underground infrastructure, especially in densely populated urban environments and geologically complex ground conditions (Li et al., 2024; Mostafa & Sousa, 2024). It is recognized that the successful execution and long-term performance of these projects naturally depend on the efficiency and structural reliability of the segmental lining systems (Han et al., 2023; Meda et al., 2016).

As the technical evolution of tunnel supports is examined, it is identified that precast concrete segmental linings are extensively utilized due to their rapid installation capabilities and high compressive strength, which enable precise assembly during tunnel advancement (Bilgin & Acun, 2024; Liao et al., 2015). These lining elements are engineered to withstand a complex combination of forces, including radial pressures from the surrounding ground, bending moments, and axial forces transmitted through the interconnected ring structure (Bickel et al., 1996; Jones, 2022). Alignment is maintained with the principles discussed by Szechy (1966), who emphasized the structural significance of the ring geometry in maintaining bore stability under varying geotechnical loads.

However, a critical motivation for optimizing these systems is observed as modern projects push into increasingly difficult geological conditions. In challenging settings, such as squeezing ground, fault zones, or regions with high groundwater pressure, segmental linings often face significant structural vulnerabilities (Abdollahi et al., 2019; Bilgin & Acun, 2024). In such adverse environments, the risk of excessive deformations and stress concentrations at the interfaces of joints is very high (Lunardi, 2008; Maidl et al., 2008).

The fundamental study carried out by Çağlar et al. (2026) based on earlier studies where the structural behavior of thinner TBM tunnel linings reinforced with internal steel rings was investigated are mentioned. Previous numerical studies have shown that proactive use of steel rings can lead to a significant reduction in concrete consumption as well as structural integrity within safe limits for some ground conditions. Therefore, an important engineering goal is proposed to extend this work to a full parametric study. The objective is to assess the performance of this hybrid support strategy in a broader range of rock mass qualities and overburden depths, thus providing a more flexible optimization framework for future TBM-driven tunneling projects.

1.2. Problem Definition

The fundamental challenge in modern TBM tunneling is identified as the inherent structural and logistical trade-offs associated with segmental lining thickness. As current design paradigms are investigated, it is observed that the primary engineering response to increasing geostatic and hydrostatic loads is a proportional increase in lining thickness (t). However, it is recognized that this conventional strategy creates a series of stepped problems that affect the entire project lifecycle.

In the analysis of the structural mechanics, it is seen that increasing the thickness of a concrete segment fundamentally alters its interaction with the ground. A thicker lining provides better axial and flexural resistance, but the alignment is consistent with the observations of Petraroia et al. (2023) who state that higher rigidity can attract higher internal forces to the lining, especially in moderately swelling or squeezing soils. Furthermore, it is acknowledged that the increasing mass of the lining tends to complicate self-weight considerations and logistical handling. As pointed out by Bilgin & Acun (2024), there are major operational challenges and risks of structural damage during installation for transporting and placing heavy and thick segments with precision in the confined space of a TBM shield.

There is also an important economic and environmental conflict identified. But thicker linings require larger excavation diameters, which may lead to increased volumes of excavated spoil and increased material consumption. According to Muir Wood (2004), the optimization of tunnel support is not a matter of strength alone, but a fine balance

between economy of materials and efficiency of construction. This is seen in deep tunneling cases where a point of diminishing returns is reached and the costs of manufacture and excavation outweigh the structural benefits of the 'concrete-only' approach.

It is observed that the structural response of the hybrid system is not static; rather, it evolves significantly under different geotechnical stress factors. To define the operational limits of the steel-ring reinforcement, the system's performance is investigated through two primary parametric variables. First, the influence of different overburden depths (H) is studied at a constant rock mass quality in an effort to understand the depth-induced stress limits. Second, the effect of deterioration of rock mass quality (GSI) is assessed at a constant depth to evaluate the robustness of the system under bad geological conditions. Table 1.1 summarizes the exact values chosen for this parametric study, which illustrates the limited range of the numerical simulations of this study.

Table 1.1 Variable Parametric Scenarios Studied

Parametric Axis	Constant Parameter	Variable Parameter	Range of Values
Depth Sensitivity	GSI = 25	Overburden Depth (H)	59, 79, 99, 119, 139 m
Geological Sensitivity	H = 90 m	Rock Mass Quality (GSI)	20, 25, 30, 35, 40
Lining Geometry	$D_i = 8.0$ m		
Thickness (t)	0.25 m		

The structured approach isolates the specific effects of depth and ground quality, so that the findings provide clear engineering guidelines for the implementation of thin, reinforced linings in diverse environments.

1.3. Purpose of the Study

The main objective of this thesis is to quantify rigorously the performance of the hybrid tunnel lining system consisting of 25 cm thin precast concrete segments combined with high-stiffness internal steel rings. The addressed research gap is how this specific setup

ensures structural integrity and safety margins in various challenging geotechnical situations. The research objectives are in line with the engineering philosophy of Mezger et al. (2017) who argued that optimization of tunnel supports must be validated through controlled analytical frameworks that isolate the effects of critical variables.

The main goal is achieved by pursuing the following specific research objectives:

- **Depth Sensitivity Evaluation:** The structural response of the hybrid system is evaluated by increasing the overburden depth (H) from 59 m to 139 m. The purpose is to find the depth where the ultimate capacity limits of t/D ratio of 1:32 is reached.
- **Analysis of Geological Resilience** Quantification of the performance of thin segments under degrading rock mass qualities where transition from fair rock ($GSI=40$) to weak rock ($GSI=20$).
- **Quantification of the "Shielding Effect":** We quantify the mechanism of load redistribution between the concrete segments and the steel rings. The composite action of the support systems is defined by (Maidl, Thewes, & Maidl, 2014) as a function of the relative stiffness of the different support elements and the surrounding rock mass. The aim is to determine the exact percentage of axial forces (N_2) and bending moments (M_{11}) that are absorbed by the steel rings. This is done so that we can understand how the high-modulus steel 'shields' the more flexible concrete sections, providing insights into the mechanical viability of the hybrid design.

Through these objectives, the structural efficiency of the strategic integration of steel rings is intended to be characterized, thereby offering a technical roadmap for optimizing TBM tunneling projects towards more sustainable and logistically efficient hybrid lining solutions.

1.4. Scope and Limitations

The scope of this research is limited to only investigating the structural behavior of a particular type of hybrid lining: 25 cm thick precast concrete segments, reinforced with strategically located internal steel rings. For the parametric evaluation, the numerical

simulations are limited to a standardized tunnel geometry with an inner diameter of 8.0 m so that they can provide a controlled environment. The methodological limits are in accordance with the framework presented by Hasanpour (2014) that emphasizes the necessity of identifying key geotechnical parameters by using simplified boundary conditions to set meaningful correlations in complex tunnel-ground interaction problems. This is the approach that has been followed. Thus, the variations in lining forces observed are directly related to the variations in depth and rock quality.

The scope of this investigation is limited to the following parameters:

- **Geotechnical Parameters:** The study is limited to rock masses with constitutive model of Hoek-Brown, with GSI ranging from 20 to 40.
- **Geometrical Variables:** The study considers overburden depths (H) from 59 m to 139 m which represents moderate to deep tunneling conditions.
- **Structural Focus:** The analysis focuses on short-term structural behavior, considering internal forces (N_2 , Q_{12} , M_{11}) and total deformations ($|u|$) immediately after equilibrium of the last staged excavation phase.

However, the following limitations should also be acknowledged to define the applicability of the findings:

- **Static Analysis:** It is acknowledged that this study is largely static. As pointed out by Abdollahi et al. (2019), dynamic tunnel responses are highly dependent on transient wave propagation and the site-specific seismic conditions, and therefore such dynamic loadings are beyond the current scope of the study.
- **Material Specificity:** To allow a direct comparison of the internal forces, linear elastic material models are used in the simulation for the lining elements (concrete and steel). While it is recognized that the non-linear material behavior presented by Meda et al. (2016) provides a more realistic prediction of post cracking performance and structural ductility, it would introduce additional complexity to the main objective of defining the initial shielding efficiency of the steel rings.
- **Time-Dependent Effects:** While geostatic equilibrium provides a baseline for long-term stability, the time-dependent effects of concrete creep or shrinkage are not explicitly modeled.

- **System Optimization and Resource Efficiency:** This study aims to characterize the structural performance and material efficiency of the thin-lining hybrid system and does not include a complete financial audit, life-cycle cost analysis (LCA) or a detailed economic comparison of installation and maintenance costs. The evaluation is restricted to the technical feasibility and load sharing efficiency of the steel reinforced configuration.

1.5. Thesis structure

The thesis is organized in five separate chapters, each one aimed to respond systematically to the research objectives set forth in this study. The structure is:

- **Chapter 1: Introduction – Background and motivation** of the study of hybrid supports for tunnels are provided. The main engineering problem is defined and the objectives, scope and limitations of the study are described.
- **Chapter 2: Literature Review –** This chapter presents a detailed review of the existing tunnel support technologies. Evolution of segmental linings and mechanical principles of composite support systems are discussed. The theoretical basis of the hybrid reinforcement strategy is the synthesis of the results of leading researchers in the field.
- **Chapter 3: Methodology and Numerical Modeling –** This chapter describes the numerical structure developed for this study based on the finite element method. This chapter presents the Hoek-Brown constitutive parameters, the staged excavation sequences and the specific parametric matrix where the overburden depth (H) varies from 59 m to 139 m and the rock mass quality (GSI) from 20 to 40.
- **Chapter 4: Results and Discussion –** The quantitative findings of the parametric simulations are presented. The reduction of internal forces in the concrete segments is analyzed in an effort to examine the impact of the “shielding effect” of the steel rings. The structural resilience of the thin-lining system under the increasing geostatic stress is discussed.

- Chapter 5: Conclusions and Future Work – This chapter summarizes the key findings of the investigation, provides final engineering recommendations for the implementation of steel-ring reinforced segments, and proposes possible directions for future research on hybrid tunnel supports.

2. LITERATURE REVIEW

2.1. Mechanized Tunneling: Overview and Current Practices

Mechanized tunneling is recognized as a highly integrated engineering system where the excavation process and the installation of the structural support occur as a continuous, synchronized operation. The motivation for use of TBMs in modern industry is the need of high advance rates and improved safety in unpredictable geological conditions. According to original theories summarized by Maidl, Thewes, & Maidl (2014), the basic efficiency of mechanized tunneling is due to the “shielding” effect, which directly protects the exposed face while the permanent segmental lining is assembled within the tail skin of the machine. The introduction of specialized machines, such as Earth Pressure Balance (EPBM) (Figure 2.1) and Slurry TBMs (Figure 2.2), and the possibility to extend the mechanized method to more and more diverse and difficult ground conditions, where face stability is maintained by active control of support pressures, is well known (Maidl et al., 2012).

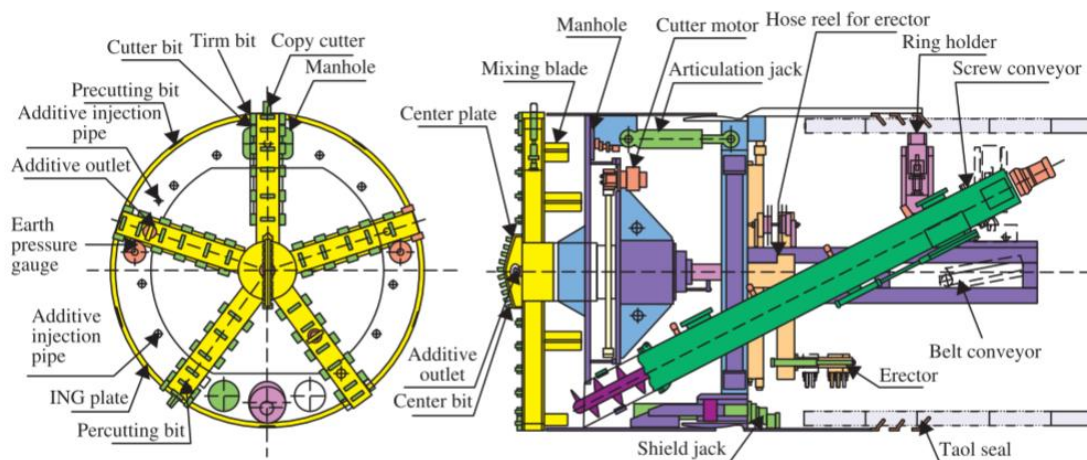


Figure 2.1 Schematic of EPBM. Adapted from (Hemphill, 2013)

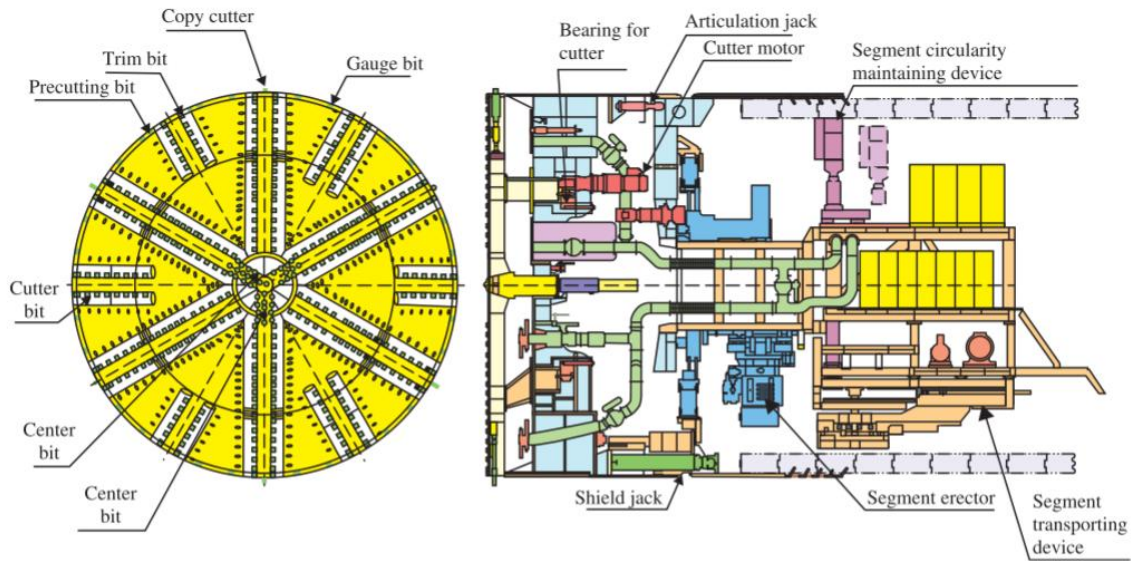


Figure 2.2 Schematic Representation of a Slurry-Type Shield Machine (Courtesy of Hitachi Zosen). Adapted from (Hemphill, 2013)

The selection of the appropriate TBM technology is fundamentally a risk management decision, where the anticipated ground conditions dictate the machine's mechanical configuration. In contemporary practice, the distinction between "open-mode" and "closed-mode" TBMs is considered critical for maintaining stability at the tunnel face, especially when dealing with rock mass qualities that exhibit high fracturing or low stand-up time. According to the research of Hasanpour (2014), operational success in such environments is not merely a matter of excavation, but of managing the "gap" between the TBM tail skin and the installed segments. This tail-void grouting process is a decisive factor in the early-age loading of the lining (Figure 2.3), where the grout pressure must be carefully balanced to prevent buoyancy or eccentric loading of the segments, as unintended pressure distributions can lead to premature structural distress (Guglielmetti et al., 2007).

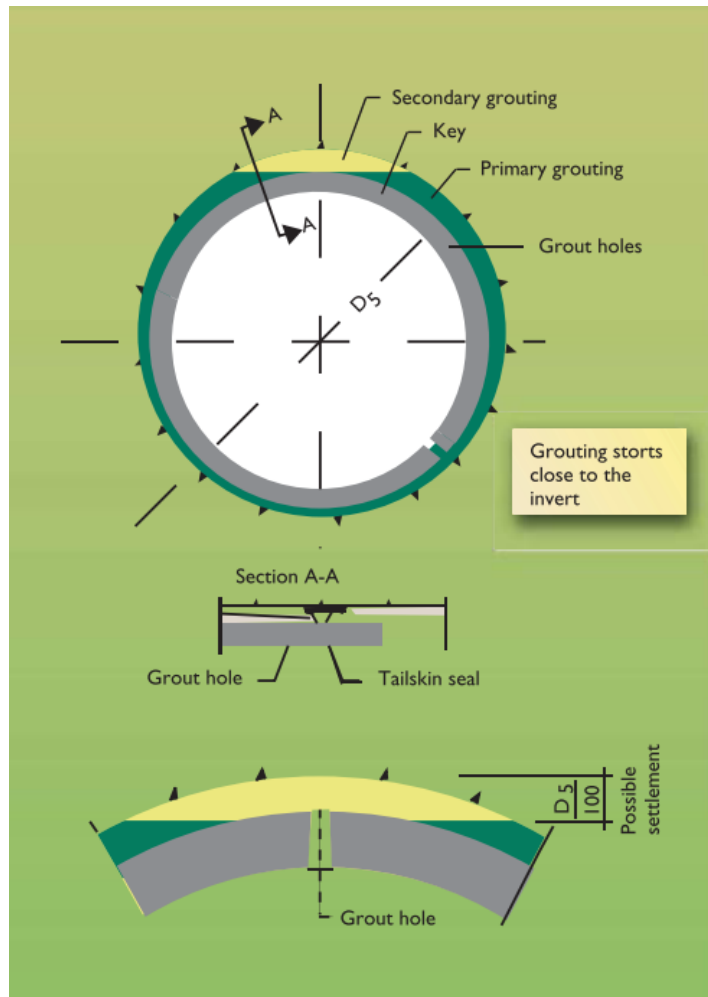


Figure 2.3 Radial backfilling through the lining. Adapted from (Guglielmetti et al., 2007)

According to Bickel et al. (1996), the “industrialization” of tunnel construction has resulted in a design culture that often increases the lining thickness to compensate for higher geostatic stresses. Increasing the lining thickness is a common practice but it is observed that this approach creates significant logistical and economic bottlenecks, especially in deep-seated alignments where the weight and volume of the segments substantially influence the project logistics and handling operations (Mezger et al., 2017). Hence, the industry is moving toward “performance-centric” optimization, by exploring new structural configurations able to withstand high geostatic pressures not only by the mass of the concrete section (Mezger et al., 2018; Petraroia et al., 2023; S. Wang et al., 2021).

Furthermore, the precast segments are manufactured with such accuracy that tolerances are within a matter of millimeters to preserve the structural integrity of the ring assembly.

It is well known that any variation in the casting process can induce considerable stress concentrations in the joints of the segments that are usually the weakest points in the lining system (Gil Lorenzo, 2021b; Maidl et al., 2008). Recent research has demonstrated that the implementation of advanced sensors and automated production lines has led to improvements in the quality control of these components, but the challenge remains the alignment of these high-precision structural elements with the inherently uncertain and heterogeneous nature of the rock mass (Maidl et al., 2008).

2.2. Segmental Tunnel Lining Systems

Segmental linings are defined as the primary structural envelope in mechanized tunneling, consisting of pre-cast concrete sections that are assembled to form a continuous support ring. It is observed that the engineering of these systems has transitioned from empirical "rule-of-thumb" designs to complex numerical characterizations that account for the non-linear interaction between the lining and the ground. According to the foundational principles discussed by Maidl, Thewes, & Maidl (2014), the segmental lining must fulfil a dual role: it serves as a permanent structural support against geostatic and hydrostatic pressures while providing the necessary reaction surface for the TBM's longitudinal thrust jacks. The geometry of these segments is generally determined by the tunnel diameter and the technical specifications of the machine. The structural performance of these segments is mainly controlled by the interaction between the lining stiffness and surrounding rock mass.

It is well known that the manufacturing of these segments is a critical stage, with high precision steel molds, to obtain dimensional tolerances within a millimeter. Specialized concrete mixes are used, usually with steel or synthetic fibers (Figure 2.4), targeted at bettering crack control and ductility and therefore segment durability (Jones, 2022; Kolymbas, 2019). In contemporary practice, the structural behavior of the assembled ring is not only a function of the concrete strength but is also heavily influenced by the longitudinal and circumferential joint configurations, which dictate the overall flexibility of the support system (Bickel et al., 1996; Gil Lorenzo, 2018).



Figure 2.4 Steel fibers and steel fiber reinforced concrete. Adapted from (Kolymbas, 2019)

As discussed by Hasanpour (2014), the design must ensure that the segments can withstand not only the final geostatic loads but also the transient stresses during the construction phase, including tail-void grouting and the potentially uneven distribution of jack forces during TBM steering. It is noted that these transient loads are often higher than the final operational loads and require a robust reinforcement strategy. Segmental linings are modular and so construction speeds can be fast but, as Bickel et al. (1996) point out, the structural efficiency of segmental linings is inherently limited by the behavior of the joints, which represent points of reduced stiffness within the overall ring configuration. Therefore, modern research is focused on optimizing the connections between segments to achieve a more homogeneous distribution of internal forces, especially in deep alignments under high-pressure conditions (Gil Lorenzo, 2018; Han et al., 2023; Liao et al., 2015).

2.2.1. Structural Behaviors

The structural behavior of segmental tunnel linings is characterized by a complex interaction of the concrete rings with the surrounding ground mass. The performance of segmental systems is controlled by the presence of longitudinal and circumferential joints which introduce discontinuities into the structural envelope, unlike monolithic cast-in-place linings. The main structural response of the lining is the redistribution of geostatic loads into axial forces and bending moments, which is strongly dependent on the relative stiffness of lining and ground (Bobet & Einstein, 2023; Muir Wood, 2004).

The structural integrity of the ring is maintained by a complex interplay between the ring's internal action and the passive resistance offered by the surrounding environment, following the analytical frameworks synthesized by Maidl, Thewes, & Maidl (2014). It has been found out that the radial displacement of the lining is determined by the stiffness and the confinement capacity of the geological strata at the deformation of lining under the vertical geostatic loads. In competent rock masses this lateral expansion is an effective activation of high confining pressures, and in weak or fractured formations the inability of the ground to provide uniform support results in an increase in ovalization (Figure 2.5). The interaction is usually modeled with the concept of “subgrade reaction”, where the ground is represented by radial springs, but it is recognized that the reliability of these models depends mainly on the accuracy of the assumed bedding conditions and the specific deformation modulus of the geological unit (Jones, 2022).

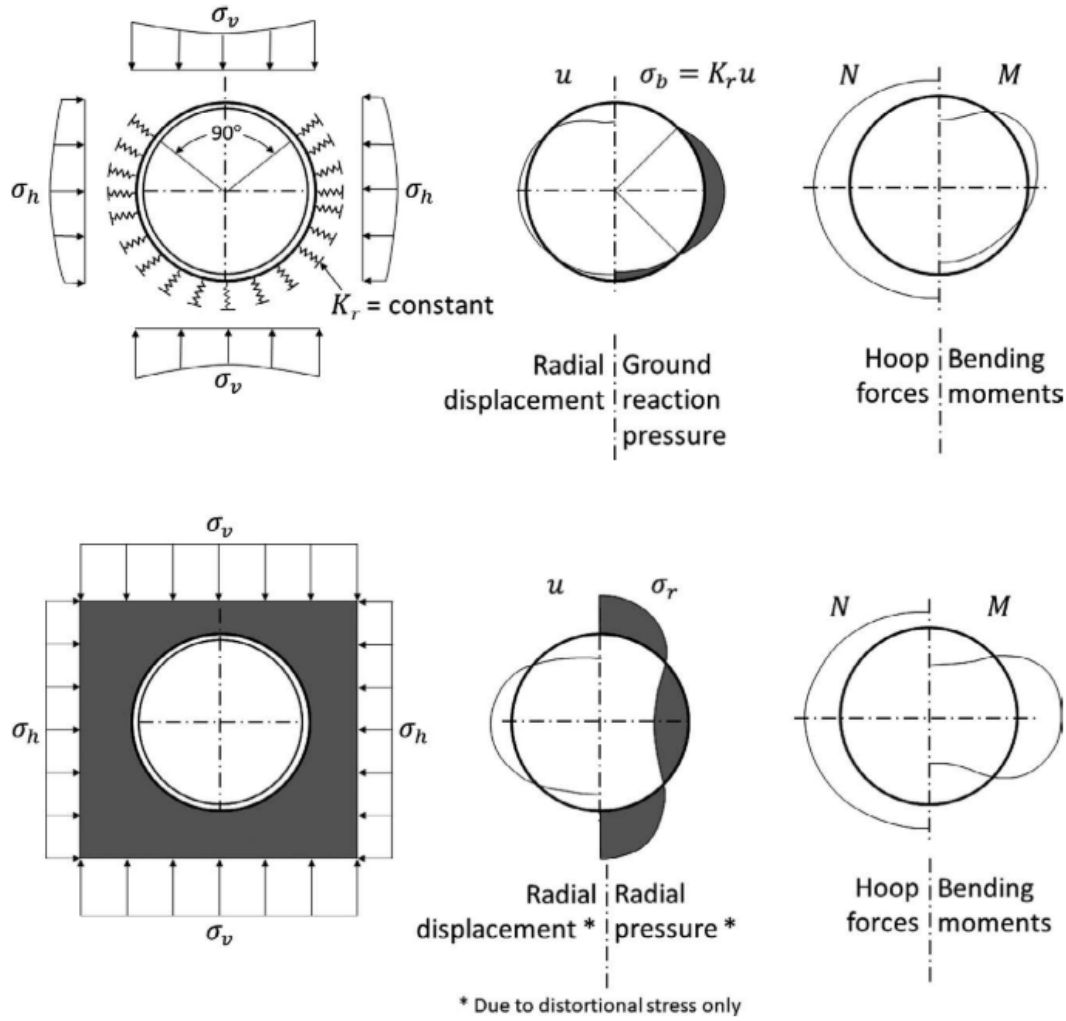


Figure 2.5 Comparison of bedded beam (top) vs continuum (bottom) analysis. Adapted from (Jones, 2022)

Furthermore, the structural behavior is significantly influenced by the "staggered" arrangement of the segments. The current practice is to rotate rings relative to each other to avoid continuous longitudinal joints, which increases the overall stiffness of the tunnel tube by the 'coupling' effect between adjacent rings (Arnau & Molins, 2015; Jones, 2022). This structural redundancy is important in the transient loading phases, when the TBM thrust jacks apply high concentrated loads to the segment faces, as noted by Hasanpour (2014). It is acknowledged that these jack forces are capable of producing local "bursting" and "splitting" stresses and the tensile strength of the concrete and the arrangement of the reinforcement in the segment joints should be thoroughly investigated.

Finally, the long-term structural behavior should consider time-dependent effects such as creep and shrinkage of concrete and gradual consolidation of the surrounding ground. It is well known that redistribution of internal forces takes place for a long time after the initial installation, especially in deep-seated alignments where the ground pressure can change due to drainage or tectonic influences. Consequently, the design of segmental linings is more and more relying on sophisticated numerical simulations that are able to account for these multi-physics interactions and ensure the serviceability of the tunnel during its design life (Bobet & Einstein, 2023; Szechy, 1966).

2.2.2. Weaknesses and Challenges

It is well known that concrete segmental linings (CSLs) are sensitive to structural performance under complex loading conditions during construction. It can be observed that a considerable amount of structural distress (e.g. cracking and spalling) are developed prior to the lining reaching the final equilibrium with the surrounding ground (Gil Lorenzo, 2021a, 2021b; Liao et al., 2015). These difficulties are mainly due to the interaction between TBM and the lining, sequential loading near the tail of the shield and unfavorable geological conditions. The longitudinal and circumferential joints, which create an intrinsic structural anisotropy, significantly reduce the global bending stiffness of the support system compared with a continuous monolithic shell (Arnau & Molins, 2015; Gil Lorenzo, 2018).

Transient stress is mainly due to the application of hydraulic jacking forces, which are required for TBM thrust and steering. Misalignment between the TBM and the lining or uneven ram loads resulting in significant eccentricity have been found to cause longitudinal cracking at the intrados of the segment and chipping at the corners (Gil Lorenzo, 2021a; Jones, 2022). The critical interaction between the jacking system and the assembled ring, which often results in localized stress concentrations, is typically visualized through the mechanical configuration of the thrust jacks and tail skin brushes (Figure 2.6). Furthermore, the structural integrity is often challenged during the transition from the shield tail to the grouted void. If the backfill grout has not yet reached sufficient stiffness, the lack of solid bedding is observed to cause the ring to ovalize under buoyancy forces or uneven pressures, leading to concrete-to-concrete contact and subsequent spalling at the joints (Guglielmetti et al., 2007; Maidl et al., 2008, 2012).

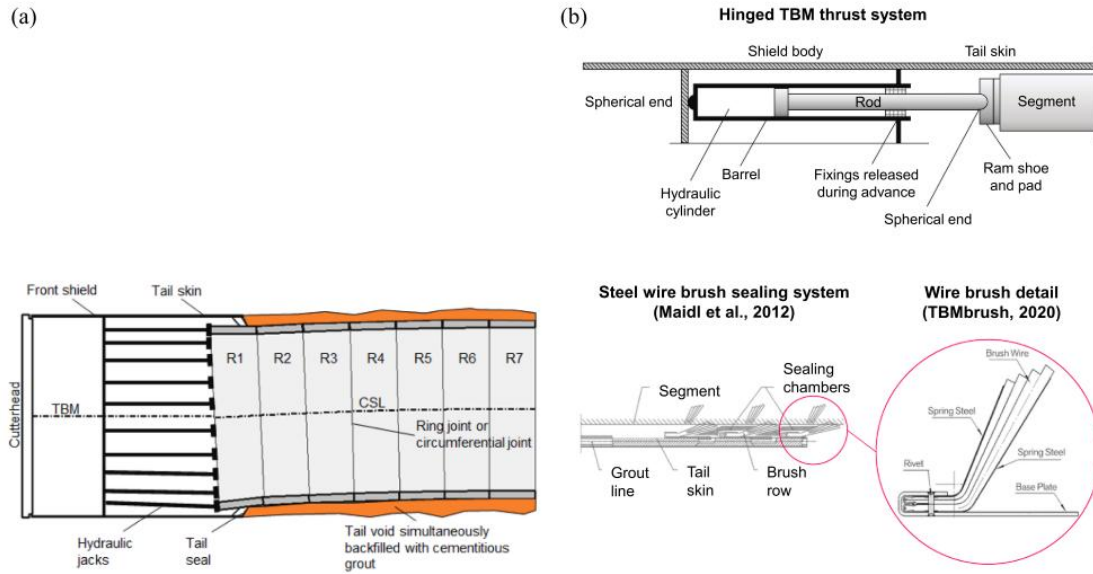


Figure 2.6 (a) Schematic representation of tunnel construction interfaces in modern mechanized shield tunnelling; and (b) details on TBM thrust system and steel wire brush system. Adapted from (Gil Lorenzo, 2021a)

Segmental joints are defined as the most critical structural discontinuities in the support system, where radial dislocation between adjacent segments is observed to cause a sharp increase in bending moments and local stress concentrations (M. Wang et al., 2019). Misalignments at these joints are also identified as a factor that compromises the compression of gaskets, which can compromise watertightness and long-term durability of the tunnel (Gil Lorenzo, 2021b; Jones, 2022). Typical damage patterns resulting from these in-plane and out-of-plane angularities, including intrados spalling and extrados torsion cracks, represent the most frequent construction-induced failures (Figure 2.7). It is shown that imperfect contact between segment rings may cause unexpected stress paths and may lead to failure under high thrust forces or external rock pressures (Gil Lorenzo, 2021a, 2021b; Maidl et al., 2012).

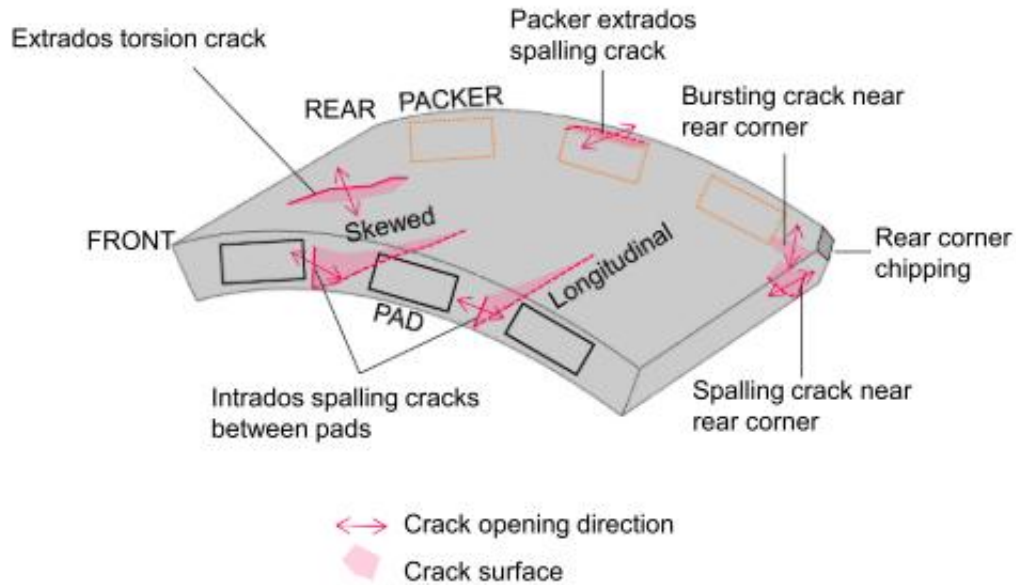


Figure 2.7 concrete damage modes in a ring identified during tail seal passage. Adapted from (Gil Lorenzo, 2021a)

In squeezing rock conditions, the high stiffness of conventional segmental linings is often viewed as a disadvantage, as these systems allow very limited convergence. Such rigidity produces high rock pressures which may exceed the ultimate bearing capacity of the lining. These types of environments require either increasing the segment thickness or the use of yielding elements to avoid structural failure, as previously found by research (Maidl et al., 2008; Mezger et al., 2018; Wu et al., 2022). Moreover, the control of ground settlements in urban areas is acknowledged as extremely important, with any failure to effectively fill the tail void or maintain stable face pressure presenting a considerable risk of excessive ground settlement (Le et al., 2023; Samadi & Hassanpour, 2024; S. Wang et al., 2021; Zhou et al., 2024).

2.3. Reinforcement Techniques in Tunnel Linings

Hence, the structural optimization of segmental linings requires a comprehensive reinforcement scheme to balance the load-carrying capacity with the operational ductility. The conventional reinforcement methods, which are basically cages of longitudinal and circumferential steel bars, are designed for the maximum bending moments and axial forces derived from geostatic calculations. Nevertheless, the use of Steel Fiber Reinforced Concrete (SFRC) has recently been included as a major improvement in mechanized

tunneling to improve the post-cracking performance and impact resistance of the segments (Gil Lorenzo, 2021b; Jones, 2022; Wagner & Schulter, 2006). The shift from conventional bar-based reinforcement to fiber-based reinforcement is a result of the localized brittle failures that are often encountered during the TBM thrust phase, where conventional bar reinforcement may not be sufficient in controlling surface spalling and corner chipping (Maidl et al., 2012; Trabucchi et al., 2021).

It is recognized that the reinforcement density and material composition choice is inherently related to the expected ground-structure interaction and the mechanical specifications of the TBM. The basic design principles dictate that the reinforcement strategies need to take into account the multi-stage loading history of the lining, from the transient stresses occurring during the segment handling and ring assembly to the long-term equilibrium states under the geostatic and hydrostatic pressures (Di Carlo et al., 2016; Maidl et al., 2008; Meda et al., 2016; Zeuli et al., 2021). Hybrid systems with steel fibers for diffused crack control and conventional bars for primary global stresses have been shown to enhance the overall structural performance of the lining. The optimization of these strategies is crucial for tunnels under deep overburden pressures, where a more robust approach is needed to ensure the serviceability and ultimate limit state performance of the lining, as discussed in recent literature (Jones, 2022; Liao et al., 2015).

2.3.1. Support Systems of NATM and TBM

The major difference between the New Austrian Tunneling Method (NATM) and TBM-based support systems is the timing and the type of the interaction between the ground and the support. The NATM support system is described as a flexible multi-stage process, where the primary lining (shotcrete, rock bolts and steel sets) is allowed to deform in a controlled way to mobilize the self-supporting capacity of the rock mass (Kolymbas, 2019; Lunardi, 2008; Mezger et al., 2018). This type of “soft” support is visualized by the classical convergence-confinement curves, which indicate that the lining is intentionally installed later, when a more favorable stress state is expected (Figure 2.8). Conversely, the TBM support mechanism is described as a “rigid” and immediate support where the precast segmental lining is placed within the tail skin of the shield and offers almost instantaneous resistance to ground movements with minimal convergence (Hemphill, 2013; Mezger et al., 2018; Schiena et al., 2024).

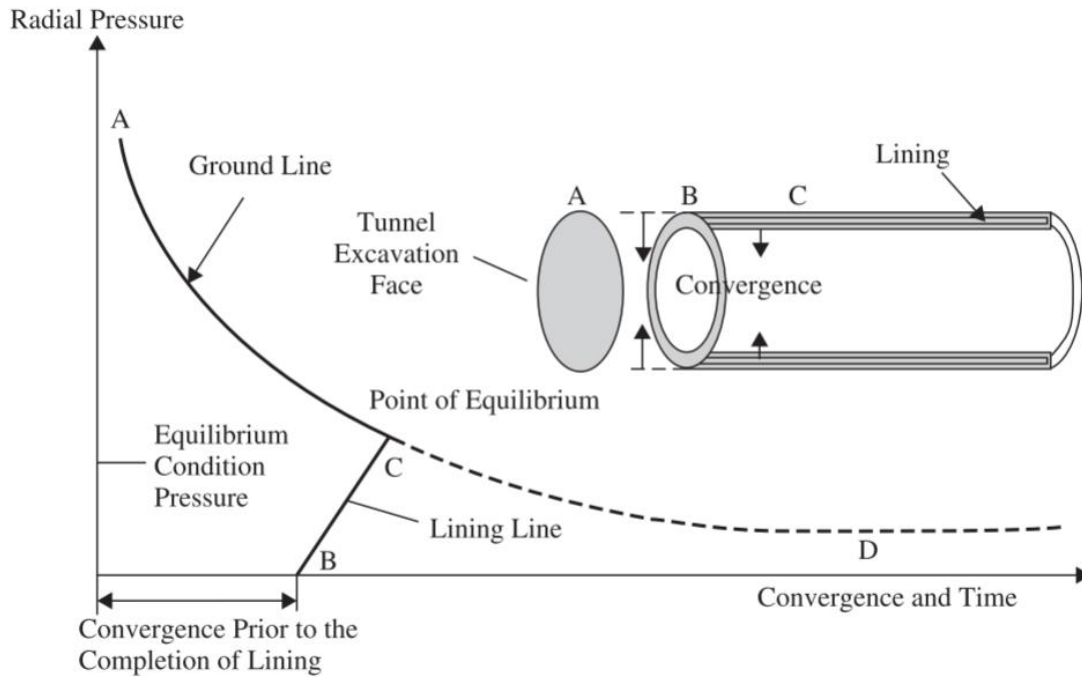


Figure 2.8 Ground-lining relationship. Re-adapted from (Hemphill, 2013)

It is observed that the mechanical behavior of TBM linings is characterized by a higher degree of structural stiffness compared to the composite shells used in conventional tunneling. While NATM relies on the redistribution of stresses through controlled yielding, TBM linings must withstand the full magnitude of geostatic pressures developed as the ground closes the tail void gap. Such rigidity makes TBM segmental linings especially vulnerable to squeezing ground conditions, where the lack of flexibility can lead to rapid pressure build-up beyond the capacity of the lining (Gong et al., 2016; Mezger et al., 2017). The design requirements for TBM reinforcement are generally more demanding than those for NATM, since they must act as a permanent structural envelope and as a reaction frame for the longitudinal thrust system of the machine (Bobet & Einstein, 2023; Maidl et al., 2008; Meda et al., 2016).

2.3.2. Steel Rings as an Auxiliary Support

Taking into account the structural deficiencies identified in deep-seated alignments, the use of steel rings as an auxiliary support system is recognized as a tactical action toward the enhancement of the radial stiffness. It is well known that these internal supports, generally made of high strength structural steel profiles, redistribute the bending moments and axial forces that would otherwise lead to an increase of the concrete thickness. Recent

methodological improvements suggested in the present research imply that the mechanical synergy of the hybrid system enables the absorption of peak geostatic pressures while the concrete segments keep the primary longitudinal stability (Çağlar et al., 2026). This interaction is effectively visualized through stress distribution models where the internal steel bracing mitigates the axial demand on the segmental lining (Figure 2.9).

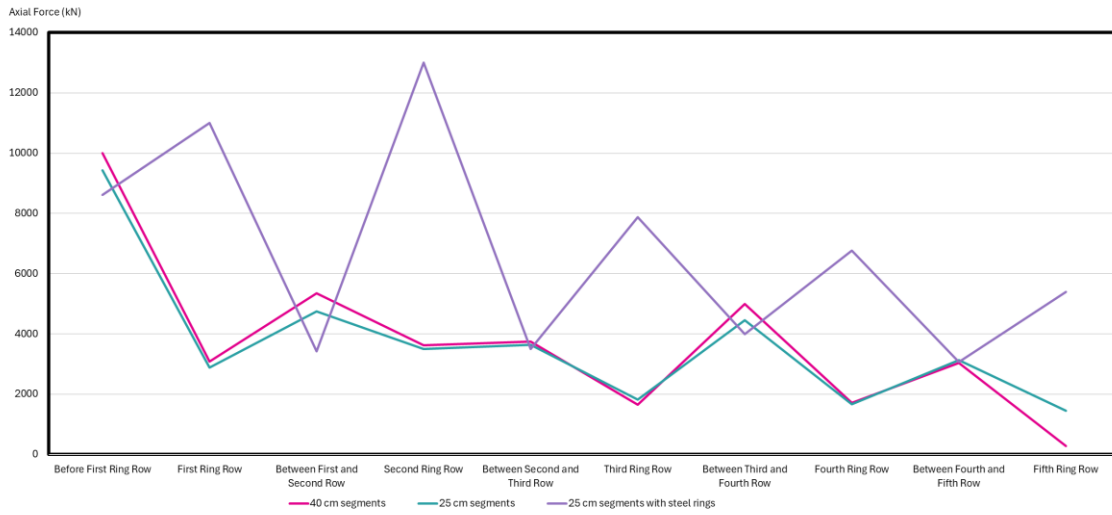


Figure 2.9 Axial stress distribution on lining with and without auxiliary steel ring supports. Adapted from (Çağlar et al., 2026).

It is noted that the use of auxiliary steel rings provides a considerable logistical advantage in that the structural capacity of the lining is separated from the volume of the concrete section. This is contrary to the traditional “mass-centric” approach, which thickens the segments and causes significant supply chain bottlenecks. Incorporating lightweight steel rings enables a substantial reduction in required segment thickness (Çağlar et al., 2026). The hybrid configuration is shown to allow the use of 25 cm thin-walled segments under high-overburden conditions, reducing the project’s logistical footprint and environmental impact while maintaining safety margins (Çağlar et al., 2026).

Moreover, the steel ring integration demonstrates a particularly high structural efficiency in squeezing rock conditions or in regions with large variability of Geological Strength Index (GSI). Concrete segments are assumed to have good resistance to compression. Internal steel bracing gives a degree of flexural ductility necessary to accommodate eccentric loading or unexpected pressure distributions during construction. Hence, the novel configuration provides a viable contribution toward 'performance-centric' design,

offering a practical alternative to over-dimensioned linings and addressing a recognized research gap in mechanized tunneling practices (Çağlar et al., 2026).

2.4. Effect of ground conditions on the lining design

The structural behavior and design optimization of segmental linings are mainly dependent on the prevailing geotechnical environment and the specific mechanical properties of the rock mass. It is established that the interaction between the lining and the ground is not a static phenomenon but is dictated by the in-situ stress state and the deformation modulus of the surrounding medium. According to the engineering principles discussed by Maidl, Thewes, & Maidl (2014), the selection of lining thickness and reinforcement layout is primarily a response to the anticipated geostatic and hydrostatic loads. It is recognized that among the various environmental factors, the Geological Strength Index (GSI) and the overburden depth are identified as the most critical parameters that determine the magnitude of bending moments and axial forces exerted on the lining assembly (Hasanpour, 2014).

The Geological Strength Index (GSI) is a major indicator to characterize the rock mass quality, which directly impacts the passive resistance and the subgrade reaction modulus (k). From the analytical framework of Hasanpour (2014), it is understood that the confinement of the segment joints is more stable due to the high GSI values of the rock masses, which is conducive to the more uniform distribution of stress. Contrary, for poor quality rock masses with low GSI, the reduction of the interlocking capacity of the ground results in larger radial deformations and higher bending moments on the lining assembly. It is understood that this GSI sensitivity requires a performance-based design, in which the reinforcement and thickness have to be tailored to the particular stiffness of the ground so as to avoid localized structural distress.

In addition to the rock mass quality, the overburden depth is recognized as the leading cause of the baseline geostatic stress state in mechanized tunneling. It is known that the magnitude of the vertical and horizontal pressures increases linearly with the increasing depth of tunnels in the rock mass. This often exceeds the independent structural capacity of traditional thin-walled segments. Bickel et al. (1996) pointed out that high overburden conditions increase the risks of squeezing behavior and hydraulic loading, imposing a heavy demand on the axial strength of the lining. It is found that in such deep-seated

alignments the traditional answer of adding concrete mass to counter these pressures creates the abovementioned logistical and economic bottlenecks. It further highlights the demand for novel auxiliary supports to maintain stability without excessive material volume.

Apart from these main factors, other secondary environmental factors are identified to contribute to the lining structural demand, although to a lesser extent in normal alignments. These factors include groundwater and pore water pressure, the orientation of rock mass discontinuities relative to the tunnel axis, and the possibility of time-dependent creep or swelling behavior under specific mineralogical conditions (Jones, 2022; Kolymbas, 2019; Zheng et al., 2025). In deep tunnels the thermal gradient and the existence of localized tectonic stresses are also recognized as further variables which might need localized adjustments in the lining design (Hemphill, 2013; Lunardi, 2008; Szechy, 1966). Thus, while these secondary factors provide a comprehensive picture of the geotechnical risk profile, the design framework of this study emphasizes the optimization of the lining system considering the dominant effects of the rock mass quality and depth-induced pressures.

2.4.1. Geological Strength Index (GSI)

The Geological Strength Index (GSI) is defined as a basic tool to characterize the reduction in strength and stiffness of a rock mass relative to the intact rock properties. Based on the fundamental frameworks developed by Hoek & Brown (1997) and refined by Marinos & Hoek (2000), the GSI system (Figure 2.10) provides a qualitative classification based on visual geological field observations. It assesses two fundamental qualitative attributes: the degree of block interlocking (ranging from intact to completely disintegrated structure) and the surface conditions of discontinuities (evaluated by roughness, weathering, and infill). Rather than relying on rigid numerical measurements, these visual observations determine an index value that serves as a direct input for the Hoek-Brown failure criterion. In mechanized tunneling, this qualitative index governs the calculation of the empirical deformation modulus of the rock mass (E_m) and the compressive strength (σ_{cm}). As emphasized by (Hoek et al., 2002), any uncertainty in GSI estimation propagates through the Hoek-Brown failure criterion, leading to significant variations in the predicted ground-structure interaction.

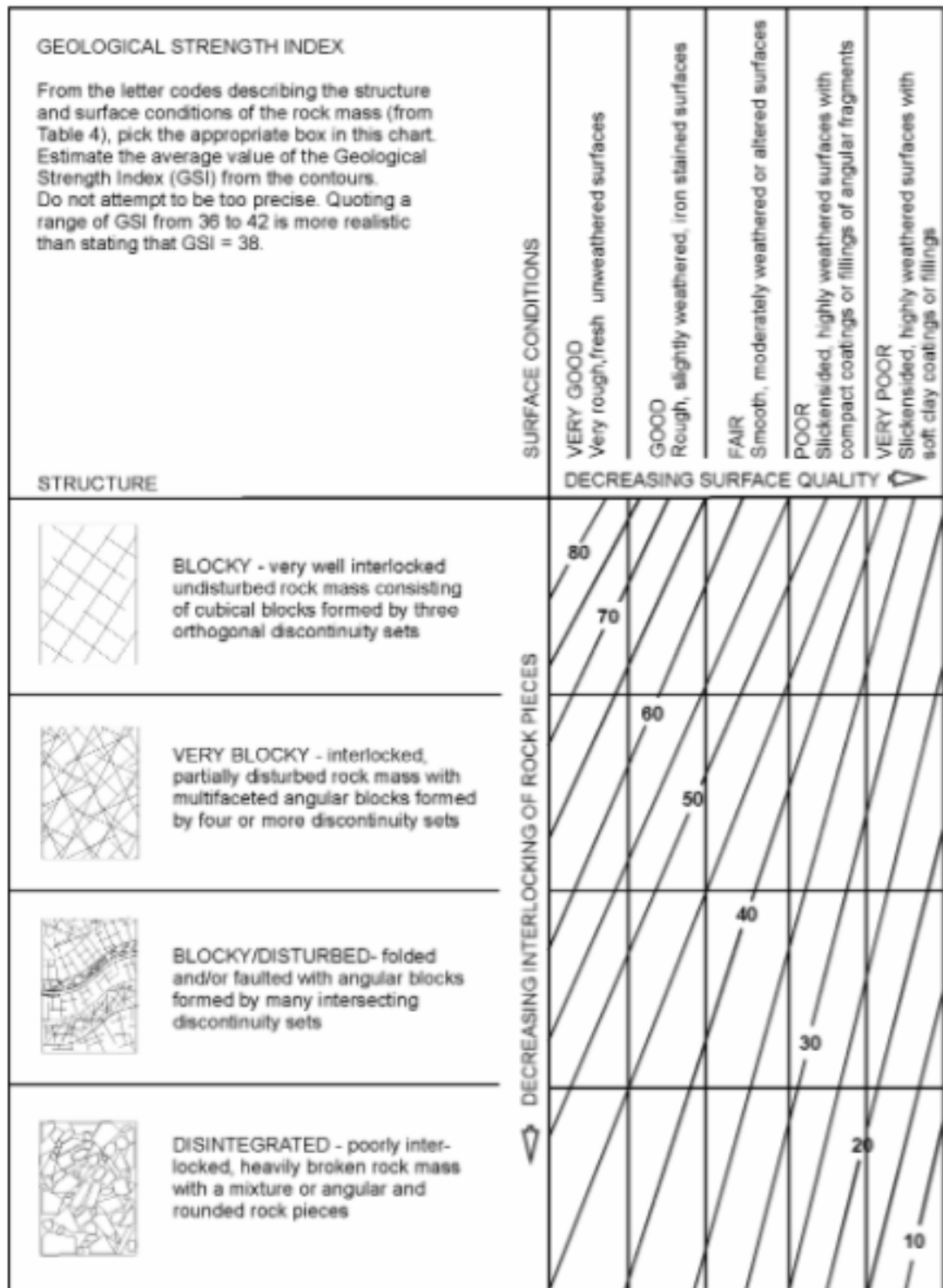


Figure 2.10 Estimate of GSI based on geological descriptions. Adapted from (Hoek & Brown, 1997)

The effect of GSI on the structural demand of segmental linings is mainly attributed to the passive ground resistance mechanism. In the analytical view of Maidl, Thewes, & Maidl (2014), a high GSI indicates a well-interlocked rock mass, which provides a stable

“bedding” for the tunnel segments and efficiently distributes the geostatic loads. However, the lack of interlocking is known to lead to a loss of self-supporting capacity in “tectonically disturbed” or “disintegrated” rock masses with low GSI values. Geological conditions of this type, as studied by Marinos & Hoek (2000), allow a plastic zone to develop around the opening, which increases the radial pressure on the lining assembly considerably.

Also, the relationship between GSI and modulus of subgrade reaction is found to be a controlling parameter in the design of thin-walled segments. In numerical frameworks, the subgrade reaction is simulated as radial springs and is directly proportional to the rock mass stiffness computed from the GSI (Hasanpour, 2014). It is established that in low-GSI environments, the reduction in ground stiffness leads to a "softening" of these springs, allowing larger convergences before equilibrium is reached. It is identified that this phenomenon is further complicated by the "anisotropy" of the rock mass, where GSI variations in different directions can lead to non-uniform loading cases that are critical (Sonmez & Ulusay, 1999).

Beyond its influence on static loading, the GSI is established as a key indicator for predicting squeezing behavior in deep tunnels. It is observed that when the ratio of the rock mass strength to the in-situ stress state is low, a condition highly dependent on the GSI, the ground undergoes time-dependent deformations that can exceed the lining's capacity. The use of auxiliary steel rings is recognized as a necessary measure in low-GSI areas to provide the necessary flexural ductility and to avoid brittle failure of concrete segments, as shown in the comparative studies of Bickel et al. (1996) and confirmed by more recent findings (Çağlar et al., 2026). Thus, the GSI is identified as the primary geological factor that requires the shift from the conventional mass-based designs to the new performance-based hybrid configurations studied in this work.

2.4.2. Overburden Depth

The overburden depth is identified as the primary driver of the baseline geostatic stress state in mechanized tunneling, where the vertical and horizontal earth pressures are recognized as a linear function of the depth of the alignment. It is established that the in-situ stress increases with the tunneling into a deeper layer of the rock mass, which has a direct impact on the load-bearing requirements for the segmental lining. According to

Terzaghi (2007) basic theories, the pressure on the lining in deep tunnels is affected not only by the immediate weight of the rock column, but also by the arching effect developed in the rock mass. In high-overburden conditions, the mobilization of this arching effect is found to be a key factor influencing whether the lining will act as a primary support or a secondary structural envelope (Bobet & Einstein, 2023; Kolymbas, 2019; Lunardi, 2008).

The effect of overburden depth on structural demand is further aggravated by hydrostatic pressure. Bickel et al. (1996) have stated that in sub-aqueous or deep-seated alignments the sum of the lithostatic and hydrostatic pressures can lead to total radial pressures that are greater than the individual compressive strength of standard concrete segments. Such phenomenon is of particular importance for thin-walled configurations where the axial capacity (N) is restricted by the reduced cross-sectional area. Maidl, Thewes, & Maidl (2014) pointed out that underestimation of the depth-induced stress gradients may result in overestimation of the segment convergence and in the occurrence of longitudinal cracks, especially when the TBM leaves the shielding of the tail skin during the grouting phase.

Furthermore, it is found that the overburden depth controls the “squeezing” behavior of weak rock masses. It is found that large time-dependent plastic deformations will occur in the ground if the ratio of rock mass strength to stress induced by the overburden is less than a critical value. In the comparative studies of Hasanpour (2014), managing these deep-seated pressures by the conventional ‘mass-centric’ approach, i.e. increasing the concrete thickness, results in significant logistical bottlenecks and economic burdens which impede the advance rate. The traditional reinforcement becomes less efficient with the increase of depth. It is argued to adopt hybrid systems based on performance.

Therefore, the employment of auxiliary steel rings is defined as a required structural evolution for deep-alignment projects in an effort to preserve safety margins without a parallel growth in material mass. Steel rings are known to provide the required flexural stiffness and ductility to withstand the high over-burden stresses, which could otherwise lead to excessive cracking or localized distress in slender concrete-only sections (Çağlar et al., 2026). Thus, the depth of overburden is not only a loading parameter but also a major logistical limitation, which justifies the novel 25 cm hybrid lining design investigated in this study, filling the research gap in deep mechanized tunneling.

2.5. Finite Element Model in Tunnel Design

The use of the Finite Element Method (FEM) is set as the foundation in modern tunnel engineering, offering a sophisticated numerical framework to simulate the complex non-linear interactions between the segmental lining and the surrounding rock mass. However, the FEM is recognized to be able to account for the geological heterogeneity, the staged excavation sequences and the structural anisotropy of the assembled ring simultaneously unlike simplified analytical closed-form solutions. The basic concepts of numerical modeling in geotechnics lay down that the primary goal of FEM is to obtain a representative equilibrium between the redistribution of ground stresses and the reaction from the support, and this allows to capture the stress-strain behavior of the system under static and transient loading conditions (Jones, 2022; Panet & Sulem, 2022; Trabucchi et al., 2021).

From a methodological point of view, the reliability of the FEM results depends on the selection of adequate constitutive models for the ground and for the structural elements. As highlighted by Hasanpour (2014), the simulation of mechanized tunneling requires a three-dimensional (3D) approach for the proper capture of the “arching effect” and the longitudinal stress transfer at the TBM face and the tail-void gap. It is found that the combination of Hoek-Brown failure criterion for rock masses and plate or shell elements for the concrete segments is the standard practice for the evaluation of the structural demand in deep-seated alignments. Furthermore, the use of “interface elements” is identified as a key requirement to reproduce the frictional behavior and possible slip between the lining and the ground, which is an important factor in determining the distribution of bending moments.

The FEM studies in the literature highlight the sensitivity of the segmental linings to construction variables. It is well known that numerical investigations are largely devoted to the influence of tail-void grouting pressure and jacking forces on the early-age stability of the segments. Finite element analyzes indicate that the asymmetric grout pressure distribution is a major factor in eccentric loading, which frequently results in structural distress not captured by conventional 2D plane-strain models, according to observations from Maidl, Thewes, & Maidl (2014). The results show that 3D FEM is the most powerful tool to detect localized stress concentrations at the segment joints, with a level of detail that is crucial for the optimization of reinforcement layouts in performance-driven designs.

In addition to the internal force analysis, FEM is increasingly used in modern FEM applications for parametric studies and sensitivity studies. It is observed that the operational envelopes of the thin-walled lining systems can be defined by the researchers by changing the key parameters such as the Geological Strength Index (GSI) and overburden depth within a numerical framework. It is claimed that these numerical results form a predictive framework for material optimization, where the use of auxiliary supports, such as steel rings, may effectively reduce the possibility of brittle fracture of concrete sections subjected to extreme geostatic pressures (Çağlar et al., 2026). In this respect, FEM is set up as a tool not only for verification purposes but as an essential design platform, allowing the transition from mass-oriented over-dimensioned linings to the novel hybrid configurations addressed in this paper.

2.6. Literature Parametric and Optimization Studies

Parametric studies and optimization algorithms are recognized as a state-of-the-art technique to improve the structural efficiency and economic viability of segmental tunnel linings. The main aim of these studies is the determination of the lining performance sensitivity to different geotechnical and operational parameters, so as to avoid rigid, over-designed configurations. Based on the systematic reviews in the literature, the initial optimization efforts have mainly focused on the concrete grade and the conventional reinforcement ratio. Early parametric studies were carried out based on simplified 2D analytical models to assess the influence of the lateral earth pressure coefficient (K_0) on the bending moment distribution. It was found that a reduction of the rotational stiffness of the lining at the joints may result in a more favorable stress distribution. These early studies established the basis for the understanding of the “flexible” behavior of segmental rings relative to monolithic linings (Jones, 2022; Maidl, Thewes, Maidl, et al., 2014).

Numerical capabilities have been further developed which allow a wider range of parametric studies, including the non-linear interaction between the geometry of the segment and complex ground conditions. Maidl, Thewes, & Maidl (2014) synthesized research indicating that parametric analyzes of segment width and thickness show that material optimization is not linear but is controlled by the “depth-to-stiffness” ratio. It is shown that the reduction of the segment thickness can improve the flexibility of the lining and allow the shed of the loads back into the rock mass, in accordance with the

convergence-confinement theory under moderate overburden conditions. However, this optimization is often lost in deep-lying alignments because of the brittleness of concrete. Thinner sections are more and more vulnerable to eccentricities imposed by tail-void grouting and TBM jacking forces.

More recently, the industry has been improving its approach to geotechnical uncertainty by incorporating statistical methods and sensitivity analyzes. It is observed that a work has been carried out by researchers on large parametric studies to quantify the effect of rock mass parameters, viz. deformation modulus and cohesion on the structural demand of the lining. It is known that these studies use the “Response Surface Methodology” to develop predictive models that can avoid thousands of individual FEA runs. These results highlight that in the “squeezing” ground conditions the classical optimization of increasing the reinforcement ratio becomes a case of diminishing returns when the self-weight of the over-dimensioned lining starts contributing to the structural distress (Mezger et al., 2017, 2018).

Recent studies have also included multi-objective optimization frameworks to trade-off the conflicting requirements of structural safety, logistical feasibility and cost-effectiveness. Hasanpour (2014) indicated that the parametric studies conducted on 3D finite element simulations have shown the Geological Strength Index (GSI) and the overburden depth as the two most significant parameters in the design space. It is recognized that optimization algorithms have been used to find the optimal “lining-to-ground” stiffness ratio for specific rock mass classes. These studies are known to have optimized conventional segmental designs for various geologies. However, a large research gap exists in the incorporation of auxiliary internal supports as a measure to realize radical material reduction in extreme environments (Bäppler, 2016; Gong et al., 2016; Li et al., 2024; Liu et al., 2016).

Recent developments in hybrid modeling have brought forward the notion of “active” vs. “passive” optimization. Passive optimization is observed to deal with material properties whereas active optimization such as incorporation of secondary steel elements deals with the structural configuration itself. More recent comparative studies have parametrically tested internal bracing systems to evaluate their use in reducing the “ovalization” of segments under high geostatic pressures. These investigations show a change to a “performance-centric” paradigm where the concrete volume is decoupled from the

structural resilience, although empirical data on segments as thin as 25 cm are scarce in the literature worldwide (Çağlar et al., 2026).

The methodology of this study is novel and overcomes the shortcomings of the previous optimization efforts. The literature has recognized the benefits of flexibility, but the lack of auxiliary ductile elements has historically limited the threshold for thinning segments. Thus, the parametric framework developed in this research constitutes an important advancement over conventional approaches. In this study, a 25 cm thin-walled segment with integrated internal steel rings is optimized by systematic variations of the GSI and overburden depth in a 3D FEA environment. This hybrid strategy presents a performance-oriented alternative where the steel rings provide auxiliary flexural capacity, addressing a limitation observed in conventional thin-lining parametric models. Thus, the findings of this thesis contribute to the scientific discourse by providing a validated operational envelope for high-capacity, low-mass lining systems in deep-seated mechanized tunneling.

2.7. Research Gap and Contribution of This Thesis

The comprehensive survey of the current literature reveals that although the design and optimization of mechanized tunnel linings have been extensively investigated by scientific research, there are still significant gaps in the structural limits of the thin-walled segments under extreme geostatic conditions. Conventional design philosophies are generally “mass-centric”, with the normal solution being an increase in concrete thickness to accommodate high overburden and poor rock mass quality. The benefits of a flexible lining are acknowledged in theory, but in practice the use of segments less than 30 cm thick for deep-seated alignments has been avoided due to the risk of brittle failure and the difficulty of eccentric construction loads.

It is identified that a critical research gap exists in the integration of auxiliary ductile elements, specifically internal steel rings, as a permanent structural strategy for segmental linings. While the literature acknowledges the use of steel sets for temporary NATM support or as secondary bracing for traditional segments, the synergic behavior of a 25 cm thin-walled hybrid system has not been comprehensively evaluated within a 3-D numerical framework. Furthermore, the operational envelopes that define the safety margins of such hybrid systems under varying Geological Strength Index (GSI) and

overburden conditions remain largely undefined. Consequently, the industry currently lacks a validated methodology to transition from logistically burdensome, thick linings to high-performance, low-mass configurations.

The thesis contributes to the scientific discourse by a systematic evaluation of the proposed 25 cm thin-walled hybrid lining system. The main contribution is the formulation of the performance-oriented optimization framework that illustrates the possibility to counteract concrete volume loss by the use of internal steel rings, thereby decoupling the structural resilience of the tunnel from its material mass. Using advanced 3-D finite element modeling, this research evaluates the operational conditions under which the hybrid configuration offers distinct mechanical and material advantages over conventional designs, especially for deep alignments with high geostatic pressures and low GSI values.

The results of this study also provide practical insights into the logistical and environmental benefits of material reduction in mechanized tunneling. The validated 25 cm configuration is shown to enable reduction of the total carbon footprint of tunnel construction and to relax the supply chain bottlenecks associated with heavy segment transportation and handling (Çağlar et al., 2026). Therefore, this work not only fills a critical void in the structural literature but also responds to the modern engineering demand for sustainable and logistically viable infrastructure in challenging geological settings.

3. MATERIALS AND METHODS

The main objective of this chapter is to present the detailed numerical framework and the methodological approach developed for assessing the structural performance of the hybrid tunnel lining configuration. The investigation is systematically structured on the basis of the finite element method (FEM) to ensure a rigorous parametric evaluation for the complex interaction between the tunnel lining and the surrounding rock mass. The exact material models, geometrical boundary conditions and constitutive parameters that drive the numerical simulations are discussed in the following sections. Moreover, the staged excavation sequences and the parametric matrix such as variation of overburden depth (H) and rock mass quality (GSI) are explicitly defined to establish a standardized and reproducible scientific workflow for the structural analysis.

3.1. Methodological Approach

The numerical investigations are carried out within a structured methodological framework that allows for the systematic isolation, quantification and assessment of the structural interactions between the hybrid lining system and the surrounding ground. The essence of this methodology is the controlled isolation of key geotechnical variables to establish clear and uncontaminated correlations. Such a method systematically limits secondary numerical artifacts and confounding variables.

This ensures that any observed variations in lining forces, such as the redistribution of axial forces (N_2), shear forces (Q_{12}) and bending moments (M_{11}), can be attributed to the systematic changes in overburden depth (H) and rock mass quality (GSI) within the established computational domain. This framework establishes a standardized computational environment, providing a solid basis for the subsequent parametric simulations and the direct evaluation of the structural shielding efficiency offered by the integrated steel rings.

3.2. Numerical Modeling Approach

In this section, the technical architecture and computational specifications of the numerical simulations are systematically presented. A special finite element procedure is employed to properly represent the three-dimensional stress redistribution around the tunnel face during excavation and structural response of the hybrid support elements. The following subsections present the selection and justification of the advanced software used, the exact spatial dimensions and geometric limits of the computed domain, and the mechanical boundary conditions that govern the model. This systematic technical approach is designed to provide a representative approximation of the mechanized shield tunneling process, maintaining computational efficiency and numerical stability.

3.2.1. Software and Justification (PLAXIS 3D)

In an effort to solve the complex 3D geotechnical problems of the tunnel boring machine (TBM) advances, the commercial finite element software PLAXIS 3D is used. This computational platform was chosen because of a number of special technical and operational features. They are necessary for advanced simulations of tunnels:

- **Advanced Structural-Geotechnical Interaction:** The software offers a strong framework to couple continuum elements (representing the rock mass) with structural elements (representing the concrete segments and internal steel rings), enabling accurate load transfer and stress redistribution at the interfaces.
- **Specialized Tunnel Designer extension.** Having the Tunnel Designer extension is an important benefit. This specialized utility provides the possibility to precisely define the geometrical circular tunnel profiles, to assign segmented lining elements and to smoothly automate complex staged excavation sequences in one coherent environment.
- **Material & Geometry Efficient Definition** The geometry interface based on parametric CAD allows systematic changes of the computational domain. Also the software is able to reproduce accurately non-linear constitutive models, which are needed to control the mechanical behavior of the rock mass and of the support elements.

- **Complete Post-Processing Capacities:** The advanced visualization tools included in the post-processing engine are able to pull out very descriptive structural force diagrams (axial forces, shear forces and bending moments) ensuring a transparent and rigorous analysis of the internal lining responses in all the parametric scenarios.

3.2.2. Model Geometry and Dimensions

Explicitly defined spatial boundaries of the finite element domain are used in an effort to avoid boundary effects while keeping computation costs reasonable. Taking advantage of structural and loading symmetry along the longitudinal axis of the tunnel, a semi-circular numerical model is adopted. This symmetry approach allows for the reduction of the computational domain by half along the $X=0$ plane (Figure 3.1). The final numerical model has the following geometric dimensions:

- **Horizontal Domain (X-axis)** The boundary extends from $X = -20$ m to $X = 0$ m, which provides a 20-meter-wide domain for the semi-circular representation.
- **Longitudinal Domain (Y-axis)** • The model extends from $Y = 0$ m to $Y = +80$ m, giving a total longitudinal length of 80 meters to cover the entire sequential advance of the TBM.
- **Vertical Domain (Z-axis):** The lower boundary of the model is held constant at a depth $Z = -25$ m. The upper boundary is dynamic, and varies between $Z = +50$ m and $Z = +130$ m. It is directly related to the fluctuations of overburden depth (H) simulated in the parametric matrix.

The excavated tunnel configuration features an internal diameter of 8.00 m and a concrete segment lining thickness of 0.25 m. In accordance with the semi-circular model design, the excavation profile is centered at the symmetry boundary. The structural trajectory of the analyzed tunnel section's center initiates at the coordinates (0, 0, -13.25) and terminates at (0, 53.5, -13.25), yielding a net analyzed tunnel advancement length of 53.5 meters within the 80-meter longitudinal domain.

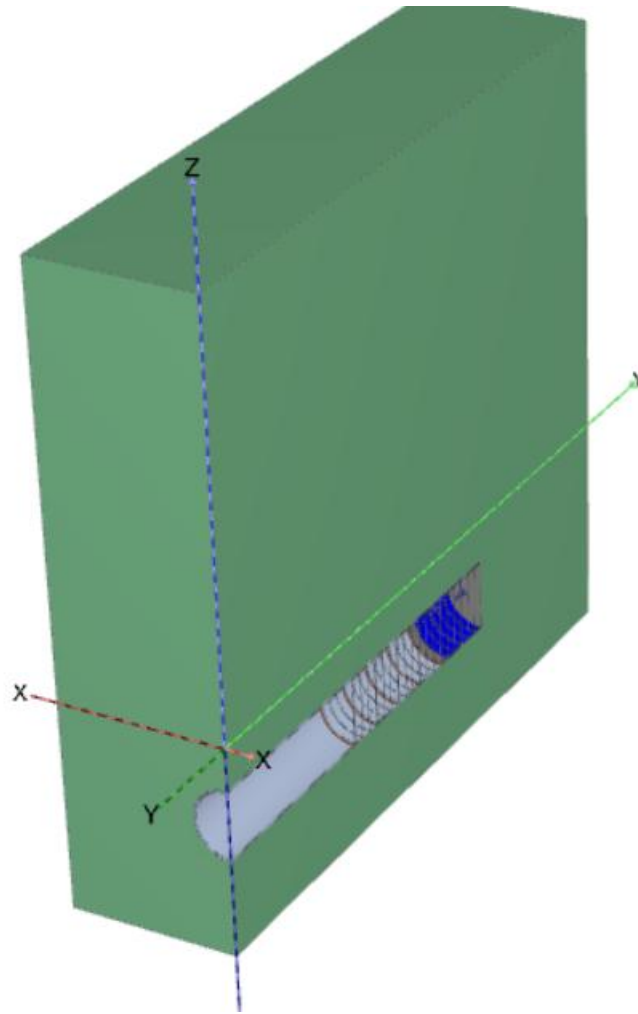


Figure 3.1 Geometric representation of the semi-circular tunnel model

Following the completion of the geometric layout, a non-uniform finite element mesh generation is executed across the computational domain. To optimize computational efficiency while ensuring high numerical accuracy in high-gradient stress zones, a systematic mesh refinement is enforced surrounding the tunnel perimeter and immediately ahead of the excavation face. The continuum rock mass domain is discretized using 10-noded tetrahedral elements, which are very effective in solving complicated three-dimensional shear deformation fields. The final finite element mesh configuration is shown in Figure 3.2, highlighting the dense element distribution at the interface of the hybrid lining and the transitioned coarse mesh toward the outer domain boundaries.

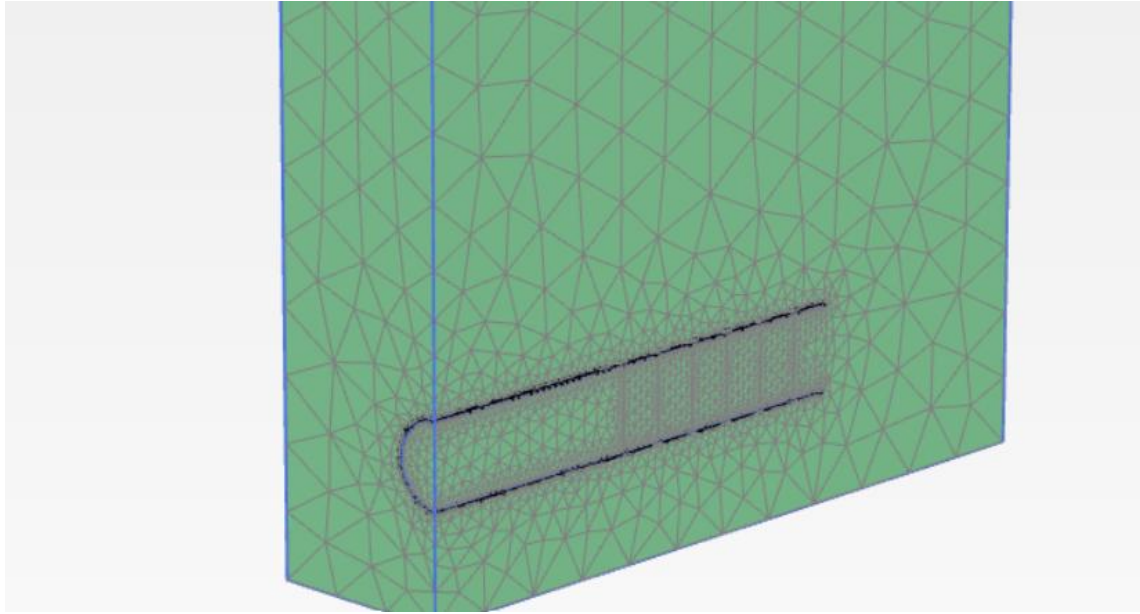


Figure 3.2 Generation of the final finite element mesh, with non-uniform discretization and local refinement of the mesh around the tunnel perimeter.

3.2.3. Initial and Boundary Conditions

Predefined boundary conditions and initial stress states are applied to the computational domain prior to excavation staging to ensure mechanical consistency and numerical stability. The mechanical boundary conditions are defined in terms of the standard deformation constraints (default fixities) within the finite element framework and are structurally detailed as follows:

- **Vertical Boundaries (X and Y-planes):** The vertical faces of the model including the longitudinal symmetry plane $X = 0$ are restricted against the displacement normal to them. Therefore, translation orthogonal to these planes is limited to translation along the Z direction only.
- **Lower Boundary (Z-plane):** The lower boundary of the numerical domain, $Z = -25$ m, is fully fixed against horizontal and vertical displacements. This prevents any rigid body translation of the model.
- **Upper Boundary (Z-plane):** The upper surface of the model represents the ground surface and is kept fully free of constraints on displacements to allow free development of tunneling induced settlements and surface deformations.

In all simulation scenarios, the hydrogeological environment is defined uniformly with the groundwater table at constant elevation $Z=0$ m. This hydrostatic condition ensures that the pore pressure distribution and the variations in effective stress are systematically taken into account throughout the parametric study.

The K_0 procedure is employed to generate the initial in-situ stress field of the undisturbed rock mass. In this analytical approach, the initial effective vertical stresses are obtained from the gravity loading of the overburden layers. The initial horizontal effective stresses are directly calculated using the earth pressure coefficient at rest (K_0). This standard initialization allows the system to reach an equilibrium state before any TBM advance is made.

3.3. Material Models and Properties

The structural behavior and the accuracy of the numerical simulations strongly depend on the realistic representation of the constitutive models and mechanical properties attributed to the ground and the support elements. In this section, the large material database used to characterize the continuum and structural components within the finite element framework is systematically described.

To describe the complex and non-linear behavior of the tunnel environment, different constitutive laws are assigned depending on the intrinsic features of each material. The elastoplastic parameters controlling the surrounding rock mass are described in the following subsections, as are the mechanical properties and cross-section properties of the concrete segmental lining and of the internal steel reinforcing rings. These definitions together form the material matrix for evaluating the structural interaction and load-sharing behavior of the hybrid support system.

3.3.1. Rock Mass Parameters (Hoek-Brown Model)

An elastoplastic Hoek-Brown constitutive model is adopted for the mechanical behavior and shear strength failure criteria of the surrounding ground. The advanced material model is used to model the non-linear strength reduction of the rock mass under progressive plastic deformations. Based on the representative geomechanical profiles, the

basic intact rock properties are characterized by intact uniaxial compressive strength (σ_{ci}) of 20 MPa and intact rock constant (m_i) of 5.

The elastic behavior of the ground domain is defined by a Poisson's ratio (ν) of 0.3. Importantly, the deformation characteristics are governed by a rock mass modulus (E'_{rm}) of 1.5 GPa, which explicitly includes the scaled geomechanical degradation and jointed character of the in-situ rock formation. While empirical formulas usually relate the deformation modulus to GSI, E_{rm} was kept constant in this study to isolate the impact of rock mass arching and strength degradation without introducing additional stiffness variables. Although a disturbance factor of $D = 0$ is typically associated with TBM excavations, a higher value of $D = 0.8$ was selected in this study as a conservative assumption. This accounts for severe stress relaxation, over-excavation gap effects, and plastic yielding around the cavity prior to grout hardening in weak rock conditions. In an effort to systematically evaluate the structural impact of varying ground conditions, the ground condition index (GSI) is introduced as a controlling variable in the failure envelope, and values of 20 and 40 are alternately considered in the parametric study. The complete set of geomechanical parameters utilized to initialize the Hoek-Brown material model is compiled in Table 3.1.

Table 3.1 Geomechanical parameters for the Hoek-Brown constitutive model.

Parameter	Symbol	Value	Unit
Intact Uniaxial Compressive Strength	σ_{ci}	20	MPa
Intact Rock Constant	m_i	5	-
Elastic Modulus	E'_{rm}	1.5	GPa
Poisson's Ratio	ν	0.3	-
Geological Strength Index	GSI	20 - 40	-
Disturbance Factor	D	0.8	-

3.3.2. Concrete Segment Properties

The precast concrete segmental lining configuration is incorporated into the finite element model using structural plate elements. The utilization of plate elements, rather than

continuum solid elements, is explicitly chosen to facilitate the direct extraction and analysis of internal structural lining forces - specifically axial forces (N_2), shear forces (Q_{12}), and bending moments (M_{11}) - developed during simulated ground-structure interaction stages.

The concrete segments are modeled as isotropic linear elastic material, which is suitable for the structural elements that are designed to operate within the elastic limit under operational design loads. The mechanical specifications initialized in the numerical model are the following: structural thickness (d) = 0.25 m and the unit weight (γ) = 24 kN/m³ of the concrete lining. The stiffness parameters governing the response of the lining are an isotropic Young's modulus ($E_1=E_2$) of 30GPa and a Poisson's ratio (ν_{12}) of 0.20. The corresponding shear modulus ($G_{12} = G_{13} = G_{23}$) is designated as 12.5 GPa. The absolute material and cross-sectional parameters assigned to the plate elements representing the concrete segmental lining are summarized in Table 3.2.

Table 3.2 Material and structural properties of the concrete segmental lining.

Parameter	Symbol	Value	Unit
Element Type	-	Plate	-
Material Behavior	-	Linear Elastic	-
Structural Thickness	d	0.25	m
Unit Weight	γ	24.00	kN/m ³
Young's Modulus	E_1, E_2	30.00	GPa
Poisson's Ratio	ν_{12}	0.20	-
Shear Modulus	G_{12}, G_{13}, G_{23}	12.50	GPa

3.3.3. Steel Ring Material Definition

The tunnel support configuration incorporates independent, full-ring structural steel segments deployed periodically along the longitudinal axis of the tunnel as a primary design component. These steel rings are put in flush and in line directly in the main lining

sequence, preserving geometric and structural continuity along the tunnel's trajectory. The support system is arranged in a systematic order in which one structural steel ring is installed after every three consecutive concrete segmental rings.

The structural steel rings have a physical ring width of 0.25 m and a structural plate thickness (d) of 0.25 m, which matches exactly the geometric cross-section and alignment of the adjacent concrete rings. These steel rings are modeled by structural plate elements in the finite element framework to represent the structural force distribution and the composite behavior in the whole hybrid support sequence.

The constitutive behavior is assumed to be isotropic linear elastic, assuming that the structural steel remains within its elastic limits under the geotechnical loads induced. Materials density is specified with an initial unit weight (γ) of 78.5 kN/m³. The mechanical stiffness is governed by an isotropic Young's modulus ($E_1 = E_2$) of 210 GPa and a Poisson's ratio (ν_{12}) of 0.30, resulting in a shear modulus ($G_{12} = G_{13} = G_{23}$) of 80.77 GPa. The absolute material and structural parameters assigned to the plate elements representing the structural steel rings are compiled in Table 3.3.

Table 3.3 Material and structural properties of the structural steel rings.

Parameter	Symbol	Value	Unit
Element Type	-	Plate	-
Material Behavior	-	Linear Elastic	-
Ring Width	-	0.25	m
Plate Thickness	d	0.25	m
Unit Weight	γ	78.50	kN/m ³
Young's Modulus	E_1, E_2	210.00	GPa
Poisson's Ratio	ν_{12}	0.30	-
Shear Modulus	G_{12}, G_{13}, G_{23}	80.769	kN/m ²

3.3.4. TBM Shield Material Definition

To simulate the temporary mechanical support and protective containment provided by the tunnel boring machine prior to the final installation of the lining sequences, the TBM

shield body is incorporated into the numerical model using structural plate elements. The modeling of the shield as a distinct structural plate ensures that the immediate stress relaxation of the excavated tunnel circumference is properly constrained during the active advancement phases.

The TBM shield is modeled as an isotropic linear elastic material with structural stiffness and mass configurations high enough to properly represent the rigid steel construction of a modern tunneling machine. The numerical thickness (d) and the equivalent unit weight (γ) of the shield plate are 0.17 m and 247.00 kN/m³, respectively, based on the calibrated database initialized in the model. The structural stiffness is defined by an isotropic Young's modulus ($E_1 = E_2$) of 200 GPa. To eliminate secondary lateral deformation artifacts within the metallic cylinder during numerical shearing, Poisson's ratio (ν_{12}) is designated as 0.00, yielding a corresponding shear modulus ($G_{12} = G_{13} = G_{23}$) of 100 GPa. The absolute material and cross-sectional specifications utilized to define the TBM shield plate are compiled in Table 3.4.

Table 3.4 Material and structural properties of the TBM shield plate.

Parameter	Symbol	Value	Unit
Element Type	-	Plate	-
Material Behavior	-	Linear Elastic	-
Plate Thickness	D	0.17	m
Unit Weight	γ	247.00	kN/m ³
Young's Modulus	E_1, E_2	200.00	GPa
Poisson's Ratio	ν_{12}	0.00	-
Shear Modulus	$G_{12} = G_{13} = G_{23}$	100.00	GPa

3.4. TBM Simulation Parameters

The realistic modeling of the mechanically driven tunneling requires an accurate quantification and application of the operational forces applied by the TBM during its advance. In this paper the computational parameters that govern the active tunneling process in the finite element model are defined in a systematic manner. Specific operational forces are applied as boundary loads to simulate the physical constraints and

stress changes on the excavation face of the tunnel and along the trajectory of the shield. The following sub-sections describe the magnitude and distribution of the face support pressure, the backfill grouting pressure applied in the tail void and the longitudinal jacking forces which drive the machine.

3.4.1. Face Pressure

To maintain the mechanical stability of the tunnel excavation face and to counteract the hydrostatic and geomechanical pressures exerted by the surrounding rock mass, a continuous face support pressure is initialized in the numerical model. This operational parameter is applied in the form of uniform perpendicular pressure acting directly on the vertical surface of the excavation face. The magnitude of this support pressure is maintained at a constant value of 750 kPa for all the computational phases and the parametric variations. This even distribution restricts stress relaxation possibilities at the tunnel core and acts as a controlled stress boundary condition directly ahead of the TBM shield advance.

3.4.2. Tail Void Grouting Pressure

To mitigate the structural relaxation of the surrounding rock mass and to minimize ground settlements immediately after the TBM shield clears the excavation zone, a continuous backfill grouting pressure is simulated within the tail void. This pressure represents the hydraulic injection of grout into the annular gap between the outer boundary of the lining segments and the excavated rock profile. This operational parameter is used in the numerical model as a uniform pressure of 500 kPa acting normal to the tunnel boundary surface. Such a perpendicular distribution of stresses prevents immediate closure of the void, ensuring early and uniform contact between the rock mass and the newly installed hybrid support sequence.

3.4.3. Jacking Forces

The hydraulic jacking forces are explicitly considered in the computational sequence to simulate the longitudinal propulsion of the TBM shield and the corresponding compressive stresses transferred to the completed tunnel support structure. This

operational parameter is implemented as an axial pressure acting exclusively onto the cross-sectional face of the newly installed tunnel lining rings. The magnitude of this thrust pressure is designated as a constant value of 4500 kPa. The load application is limited to the structural lining profile that mimics the physical effect of the thrust cylinders pressing against the concrete and steel rings to push the machine forward and thus allows the development of realistic longitudinal stress fields within the hybrid support system.

3.5. Parametric Study Design

A detailed parametric study is systematically planned to identify, assess and quantify the structural performance, load sharing capacity and overall geomechanical efficiency of the proposed hybrid support system. In mechanically driven tunneling, the structural response of the lining components is fundamentally governed by a complex interaction between the depth of the excavation and the inherent quality of the surrounding rock mass. To investigate these interdependencies under highly controlled numerical conditions, a multi-variable computational matrix is developed.

The primary objective of this parametric framework is to subject the sequential concrete and structural steel rings to a realistic spectrum of geotechnical stress states. By systematically changing the structural demand and the ground strength characteristics, the boundary condition for the active participation of the structural steel rings in the stress redistribution can be mapped out. The following subsections describe the selection criteria and engineering reasons for the independent variables that are investigated and, subsequently, the formalized presentation of the calculated unified simulation matrix configurations within this research.

3.5.1. Variable Parameters

The parametric analysis is based on two primary independent variables to systematically isolate individual and coupled impacts of geomechanical ground quality and overburden-induced stress fields on the hybrid support configuration: Geological Strength Index (GSI) and overburden depth (H). These variations are then divided into two different series of simulations where one parameter is kept constant and the other one is varied in steps.

In the first simulation series, the spatial stress state is standardized by enforcing a constant overburden depth (H) of 99 m across all models. Under this baseline gravitational stress field, the quality of the surrounding rock mass is incrementally varied across a spectrum representing weak to intermediate geomechanical conditions. The GSI values are escalated from 20 to 40 in uniform steps of 5 ($\text{GSI} = 20, 25, 30, 35, \text{ and } 40$). This setup isolates the structural response and load-sharing transitions of the sequential concrete and steel rings as a direct function of changing ground competence under a fixed depth scenario.

In the second simulation series, the intrinsic quality of the rock formation is held strictly constant at a fixed GSI value of 25. Concurrently, the structural demand is progressively intensified by varying the overburden depth (H) from shallow-to-deep tunneling regimes. The depth parameter is incrementally increased over a range from 59 m to 139 m at consistent intervals of 20 m ($H = 59, 79, 99, 119, \text{ and } 139$ m). This systematic variation allows to assess the structural capacity limits and performance stability of the hybrid lining under increasing anisotropic stress fields, ensuring a direct baseline intersection with the first series at the 99 m depth configuration.

3.5.2. Table of Scenario Matrix

In an effort to provide a well-structured and easily navigable overview of the processed computation models, the simulation matrix is split into two distinct operational tables, based on the particular independent variable isolated. The common geomechanical link and reference datum to both parametric series is the baseline model configuration (overburden depth, $H = 99$ m; Geological Strength Index, $\text{GSI} = 25$).

Table 3.5 outlines the first series of numerical setups, which isolates the structural response of the hybrid lining system as a direct function of degrading rock mass quality under a constant gravitational stress field. Conversely, Table 3.6 catalogs the second series, where the structural demand is progressively intensified by scaling the overburden depth while keeping the intrinsic ground properties uniform.

Table 3.5 Parametric Simulation Series I – Influence of Geological Strength Index (GSI) at constant depth.

Scenario ID	Overburden Depth, H (m)	Geological Strength Index, GSI	Operational Condition / Description
S1-G20	99 (<i>Constant</i>)	20	Weak Rock Mass Regime
S1-G25 (Ref.)	99	25	Mutual Baseline Reference Model
S1-G30	99 (<i>Constant</i>)	30	Fair Rock Mass Regime
S1-G35	99 (<i>Constant</i>)	35	Fair-to-Intermediate Rock Mass Regime
S1-G40	99 (<i>Constant</i>)	40	Intermediate Rock Mass Regime

Table 3.6 Parametric Simulation Series II – Influence of Overburden Depth (H) at constant ground quality.

Scenario ID	Overburden Depth, H (m)	Geological Strength Index, GSI	Operational Condition / Description
S2-H59	59	25 (<i>Constant</i>)	Shallow Tunneling Regime
S2-H79	79	25 (<i>Constant</i>)	Intermediate-Shallow Tunneling Regime
S1-G25 (Ref.)	99	25	Mutual Baseline Reference Model
S2-H119	119	25 (<i>Constant</i>)	Slightly Deep Tunneling Regime
S2-H139	139	25 (<i>Constant</i>)	Deep Tunneling Regime

3.6. Staged Excavation and Construction Simulation

To capture the three-dimensional stress redistribution, progressive ground relaxation, and the structural interaction between the advancing TBM and the hybrid support system, a rigorous staged construction sequence is established in PLAXIS 3D. The numerical simulation replicates the cyclic nature of mechanized tunneling by systematically shifting boundary pressures, structural activations, and volumetric excavations along the longitudinal trajectory (Y-axis) of the model.

The forward computational sequence is arranged in consecutive steps that correspond to the physical dimensions of the structural support members. The steps of the excavation are dynamically adapted to the width of the lining ring installed in this phase. For the normal concrete supports, longitudinal excavation step of 1.5 m is used, which corresponds directly to the concrete segment ring width. In contrast, in the computational steps concerning the insertion of the internal structural reinforcement, the excavation step is reduced to 0.25 m to correspond directly with the geometric width of the structural steel rings.

For each numerical phase, the modeling sequence performs sequentially the following operations:

1. **Volumetric Excavation and Face Support:** The soil/rock core inside the current tunnel slice (1.5 m or 0.25 m depending on the cycle) is deactivated to simulate the physical removal of the ground. Meanwhile, the uniform tunnel face pressure (750 kPa) is applied to the newly exposed boundary of excavation to maintain the stability in front of the shield.
2. **Shield Advancement:** The temporary TBM shield plate element is deployed into the newly excavated zone to provide immediate, rigid radial containment to prevent unconfined relaxation of the rock mass. At the same time, the jacking forces (4500 kPa) are transmitted axially to the face of the cross section of the previous lining ring.
3. **Lining Activation and Grouting:** The permanent lining elements are activated in the step directly after the advance of the TBM shield. Depending on the location in the 3+1 hybrid architectural sequence, a 1.5 m concrete segmental ring or a

0.25 m structural steel ring is initialized. These structural elements are activated just behind the trailing edge of the shield tail. At the same time, the uniform tail void grouting pressure (500 kPa) is applied perpendicularly to the outer perimeter of the newly installed support ring to simulate the hydraulic injection of the backfill grout into the annular gap.

Repeating this staggered computational cycle over the whole length of the numerical domain, the progressive transfer of ground loads from the rock mass to the TBM shield and finally to the unique sequence of alternating concrete and steel support rings is rigorously evaluated.

3.7. Performance Metrics (Key Performance Indicators - KPIs)

A specific set of Key Performance Indicators (KPIs) is established to quantitatively evaluate, compare and define the structural integrity and the load sharing efficiency of the sequential hybrid lining configuration under varying geomechanical parameters. These performance metrics serve as the primary engineering criteria to determine the operational limits of the composite concrete-steel support sequence. The mechanical responses extracted from the numerical simulations are systematically evaluated using the structural indicators detailed in the following subsections.

3.7.1. Total Displacement

The overall kinematic response of both the surrounding rock mass and the structural lining sequence is monitored through total displacement vector fields. This metric considers the absolute spatial displacement due to the progressive relaxation of the ground right behind the TBM face excavation. The total displacements at critical structural nodes serve as an indicator of the lining assembly's response in restraining tunnel convergence and evaluating ground movement trends.

3.7.2. Axial Forces (N_2)

The axial longitudinal forces (N_2) are used to determine the structural thrust and hoop stresses developed in the individual support rings. In 3D plate formulations this structural

metric is the normal stress integrated over the lining cross-sectional area acting circumferentially along the ring profile. To quantify the progressive load-sharing mechanism and verify that the compressive structural demand does not induce premature material crushing or buckling instabilities, it is important to monitor the distribution of N_2 over the alternating sequences of concrete segments and structural steel elements.

3.7.3. Shear Forces (Q_{12})

The structural transfer of the asymmetric ground loads and the formation of localized shearing zones possible over the lining joints are studied through the structural shear forces (Q_{12}). This parameter is a measure of the out-of-plane transverse shear stresses of the structural plate sections. High shear concentrations are usually present in zones of high geomechanical anisotropy or at the stiff interfaces between the flexible concrete parts and the rigid internal steel rings, so Q_{12} is an important parameter to evaluate the risk of structural tearing or joint failure.

3.7.4. Bending Moments (M_{11})

The flexural structural behavior and the ovalization tendencies induced in the circular tunnel geometry are captured by the distribution of bending moments (M_{11}). The metric is used to control the internal bending stresses caused by uneven radial ground pressures around the perimeter of the lining. The maximum positive and negative values of (M_{11}) are systematically mapped, so as to identify the locations of maximum development of tensile stress, which dictates the requirements of detailing of the structure and the cracking thresholds of the segmental concrete matrix and of the structural steel reinforcements.

3.7.5. Other KPIs

To establish a holistic numerical verification framework for the hybrid 3+1 lining architecture, the structural tracking is extended beyond conventional cross-sectional metrics. Evaluating the composite performance under high overburden depths requires a multi-parametric longitudinal and out-of-plane analysis of internal stress fields and

localized directional actions. Therefore, the following main structural parameters are explicitly defined and monitored in the post-processing domain for the evaluation of structural integrity and load-sharing efficiency:

- **Longitudinal Axial Force (N_1):** It is the normal force parallel to the longitudinal axis of the tunnel. The variation of N_1 must be monitored to evaluate the longitudinal continuity, the coupling stiffness, and the stress transfer between adjacent structural ring sequences, which are driven by sequential TBM thrust jacks and progressive three-dimensional ground deformation around the shield.
- **In-Plane Torsional Moment (M_{12}):** The measure M_{12} , which signifies the in-plane torsional moment developing in the lining elements. Tracking M_{12} is driven by local shear stress differentials, anisotropic ground relaxation and directional driving corrections of the TBM. It helps to keep the structural segment boundaries with enough torsional resistance without localized concrete spalling.
- **Out-of-Plane Bending Moment (M_{22}):** This structural parameter measures the bending moments acting in the longitudinal direction of the tunnel. M_{22} shows significant variations due to non-uniform longitudinal ground settlement and step-by-step ring erection sequences at high overburden depths, making this parameter essential for assessing the longitudinal structural bending distress across ring joints.
- **Out-of-Plane Shear Force (Q_{13}):** The Q_{13} metric tracks the transverse shear force acting perpendicular to the lining plane along the longitudinal axis. This parameter is highly sensitive to the sudden stiffness changes at the junctions between structural steel rings and standard concrete panels, and monitoring it ensures that longitudinal punching shear limits are not exceeded.
- **Out-of-Plane Shear Force (Q_{23}):** This parameter represents the circumferential transverse shear force that is perpendicular to the lining plane. Tracking Q_{23} variations under severe vertical to horizontal geostatic stress ratios in deep environments ensures the safety of the composite structural matrix in terms of localized out of plane punching shear failure thresholds.

4. RESULTS AND DISCUSSION

This chapter presents and evaluates the numerical outputs obtained from the comprehensive three-dimensional finite element simulations executed for the hybrid 3+1 tunnel lining architecture. The primary objective is to document the structural performance trends and directional demand shifts across the key performance indicators (KPIs) established in the methodology. The variations in internal lining forces, moments, and deformations are monitored under structured combinations of geological quality and operating overburden depths to map the mechanical behavior of the composite support assembly.

The primary datasets and numerical extraction profiles utilized throughout this chapter are evaluated across two main parametric frameworks. The first framework describes the structural response variations along five different deep alignment depths namely scaling through $H = 59$ m, $H = 79$ m, $H = 99$ m, $H = 119$ m and $H = 139$ m. The second framework considers variations in performance in five progressive geomechanical regimes and scales through different Geological Strength Index values from weak to intermediate ground conditions, $GSI=20$, $GSI=25$, $GSI=30$, $GSI=35$ and $GSI=40$. Rather than providing isolated cross-sectional interpretations, the results for each specific KPI are structured to sequentially present the numerical outputs and directly discuss the mechanical influences of both depth scaling and geological competence together within their respective sub-sections.

To eliminate redundant data representation, the structural evaluation focuses on documenting macroscopic physical trends, percentage-based demand shifts, and the spatial displacement of peak force actions along the lining perimeter. The numerical trends detailed herein are directly supported by corresponding graphical plots and integrated data tables. By synthesizing the numerical observations directly with discussions on ground arching, three-dimensional stress redistribution, and the shielding efficiency of the alternating stiffness system, this chapter establishes a coherent foundation for assessing the structural viability and safety factor verifications of the hybrid lining layout.

4.1. Total Deformation (u)

The structural performance of the hybrid 3+1 tunnel lining architecture is documented through the spatial distribution and magnitude shifts of total deformations across the sequential ring rows. The numerical parameters record these displacement trends across different geological qualities and operating overburden depths along the tunnel alignment.

When evaluating the parameters of the rock mass quality, a continuous upward trend in total deformation magnitudes is captured as the Geological Strength Index value decreases. The numerical metrics for this geological scaling across all examined states are detailed in Table 4.1, while the corresponding longitudinal displacement trajectories are illustrated in Figure 4.1. In the ground scenario characterized by a GSI value of 40, the overall deformation metrics along the monitored coordinates remain minimal.

However, as the ground quality progressively drops through the intermediate stages down to a GSI value of 20, the total deformation experiences a direct expansion. As detailed across the data columns of Table 4.1, the peak displacement metrics more than double when moving from the highest GSI baseline to the lowest GSI classification. To document the structural movement limits of the lining assembly under the most demanding geomechanical state without redundant layout replication, the cross-sectional distribution patterns and boundary vectors of these radial movements are explicitly mapped in Figure 4.2 specifically for the critical scenario where GSI equals 20.

Simultaneously, a distinct chronological and spatial trend is captured when tracking the deformation behavior along the sequential ring rows behind the active TBM excavation drive. The structural components positioned at the fifth ring row and the immediately adjacent joint interfaces consistently record the absolute maximum deformation values across all simulated ground profiles in Figure 4.1. As the tracking path moves continuously backward from this section toward the older support sequences at the first ring row and the isolated sections preceding it, a steady and monotonic reduction in total deformation values is recorded. This downward trend is systematically documented across the row sequences of Table 4.1.

Furthermore, the continuous displacement tracking along the tunnel alignment shows a periodic magnitude fluctuation between the alternating support components. As illustrated by the wave trajectories in Figure 4.1, the total deformations recorded at the

specific coordinates of the structural steel rings are smaller than the displacement values tracked within the adjacent, standard precast concrete segment zones. The intensity of this periodic fluctuation varies depending on the GSI value. In ground conditions where GSI equals 40, the deformation curve remains nearly uniform across both steel and concrete coordinates. Conversely, as the GSI value drops to 20 in Table 4.1, the periodic deformation differences between the neighboring steel and concrete elements expand, which is also visually represented in the cross-sectional profile of Figure 4.2.

Table 4.1 Total Deformation Values for GSI Parametric Scenarios

u (mm)	S1-G20	S1-G25	S1-G30	S1-G35	S1-G40
Before First Ring Row	12	12	11	9	10
First Ring Row	21	17	15	15	15
Between First and Second Row	28	22	19	16	15
Second Ring Row	23	20	17	16	14
Between Second and Third Row	36	28	23	19	16
Third Ring Row	34	27	22	18	16
Between Third and Fourth Row	43	31	26	21	19
Fourth Ring Row	38	27	23	18	16
Between Fourth and Fifth Row	44	31	26	22	18
Fifth Ring Row	42	30	25	21	18

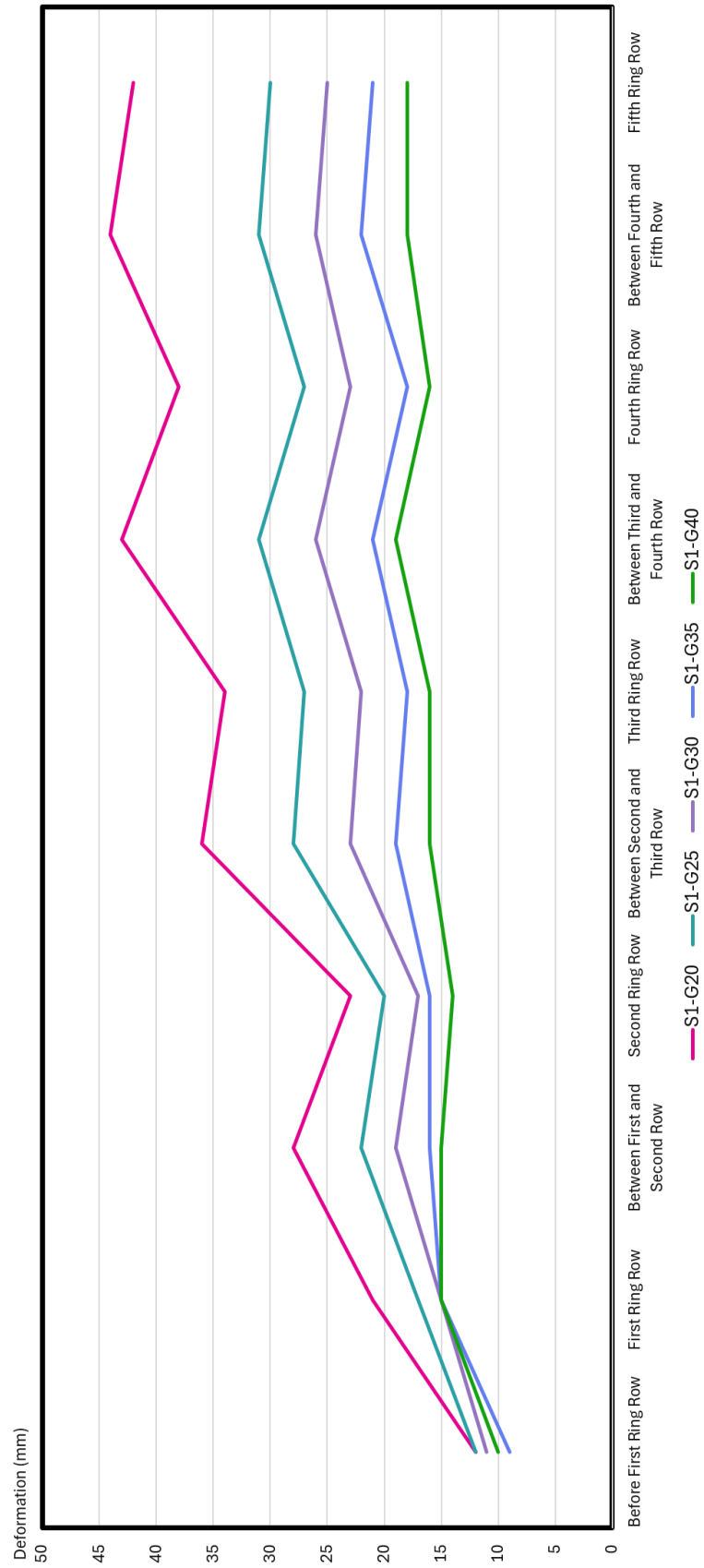


Figure 4.1 Longitudinal distribution of total structural lining deformations across varying GSI configurations at a constant H of 99 m.

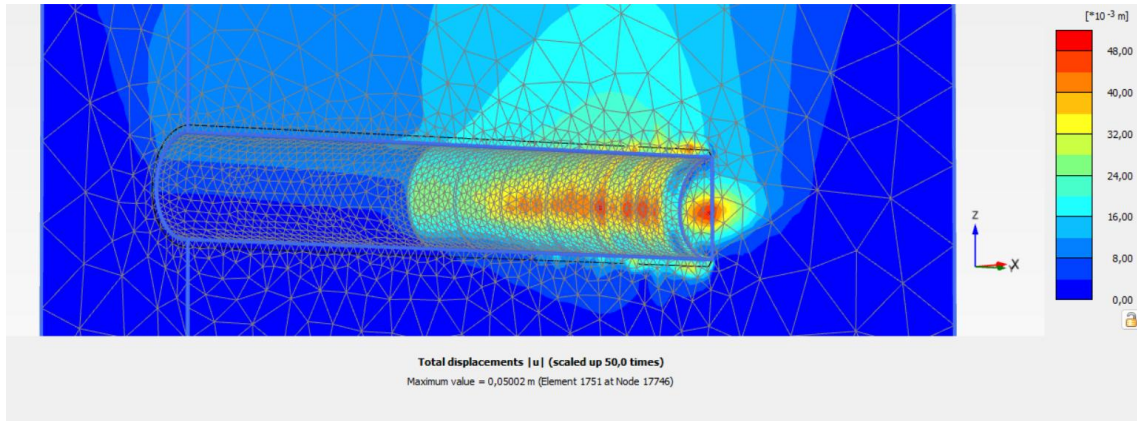


Figure 4.2 Three-dimensional finite element contour distribution of total geomechanical displacements ($|u|$) along the active excavation drive for the baseline reference model (GSI = 20, H = 99 m).

A parallel parametric trend is recorded when evaluating the independent variations of the operating overburden depth. The numerical progression for this depth-based configuration is detailed in Table 4.2, and the resulting spatial displacement variations along the lining assembly are plotted in Figure 4.3. As the design configuration increases through the five progressive depth stages from H = 59 m, through H = 79 m, H = 99 m, and H = 119 m, up to H = 139 m, the absolute magnitudes of the total structural deformations undergo a proportional expansion across all monitoring stations. The highest depth configuration of H = 139 m generates the absolute maximum displacement limits recorded across the parametric matrix, as documented in Table 4.2.

The longitudinal behavior across the ring rows remains consistent under depth scaling, where the total deformations undergo a systematic reduction when moving from the youngest fifth ring row backward to the first ring row and the stable zones preceding it. However, the variation of the periodic displacement differences between neighboring elements shows distinct profiles across the depth stages. Under shallow alignment conditions characterized by H = 59 m in Table 4.2, the deformation profile along the tunnel axis remains uniform and nearly identical across both steel and concrete coordinates in Figure 4.3.

As the overburden depth scales upward toward H = 139 m, the differences between the alternating layout components become prominent. As shown by the peak-to-valley drops along the maximum depth curve in Figure 4.3, the steel rings restrict localized displacements, whereas the adjacent concrete panels show greater relative movement, thereby causing the total deformation differences between neighboring structural

components to widen at deeper alignment boundaries. To focus the visual presentation on peak demand distributions under the most demanding vertical loading condition, the global cross-sectional displacement contours are explicitly visualized in Figure 4.4 specifically for the maximum deep overburden scenario where H equals 139 m.

Table 4.2 Total Deformation Values for Depth Parametric Scenarios

u (mm)	S2-H59	S2-H79	S2-H99	S2-H119	S2-H139
Before First Ring Row	6	9	12	15	18
First Ring Row	15	16	17	20	26
Between First and Second Row	16	19	22	27	36
Second Ring Row	16	17	20	22	25
Between Second and Third Row	17	22	28	38	53
Third Ring Row	18	22	27	36	51
Between Third and Fourth Row	20	25	31	43	64
Fourth Ring Row	17	22	27	36	53
Between Fourth and Fifth Row	19	24	31	43	69
Fifth Ring Row	18	24	30	39	55

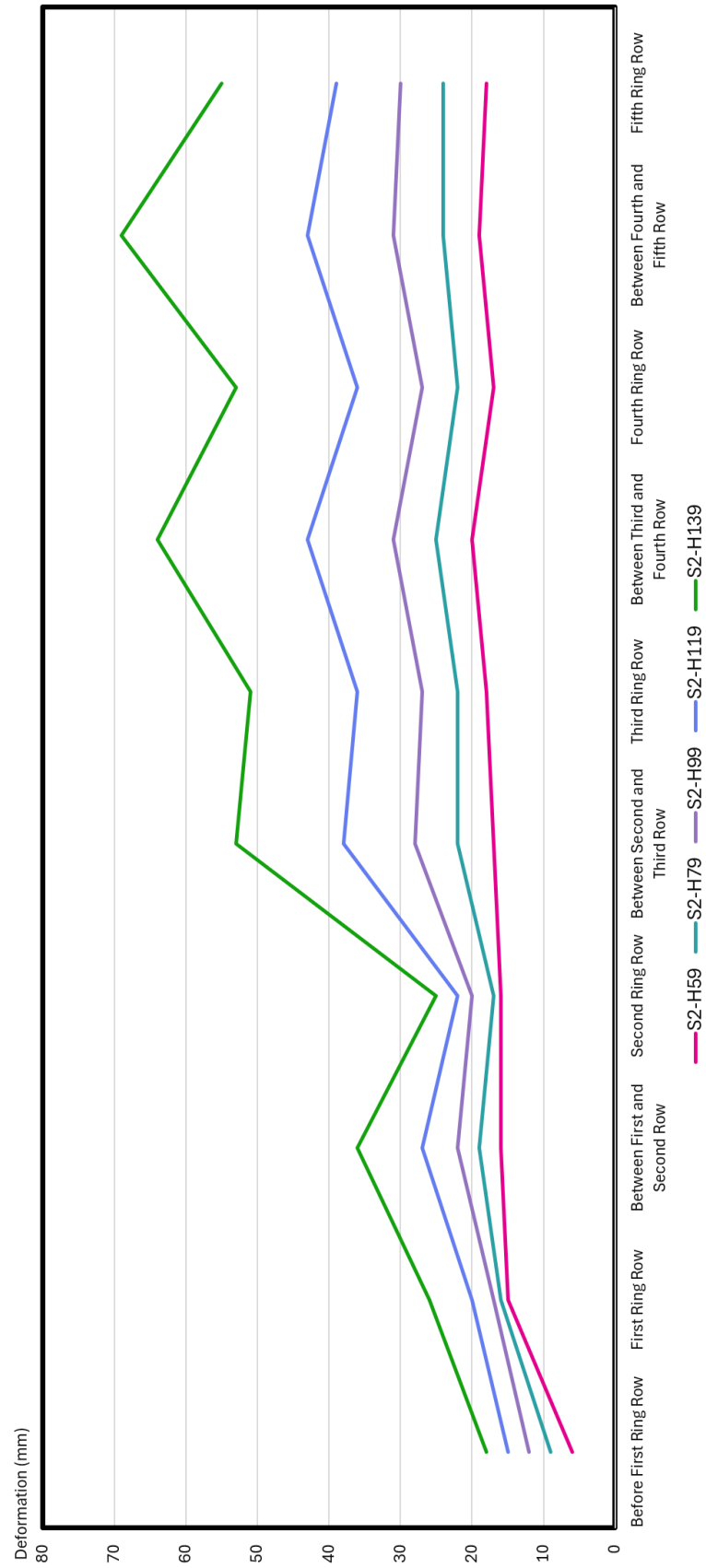


Figure 4.3 Longitudinal distribution of total structural lining deformations across varying H configurations at a constant GSI of 25.

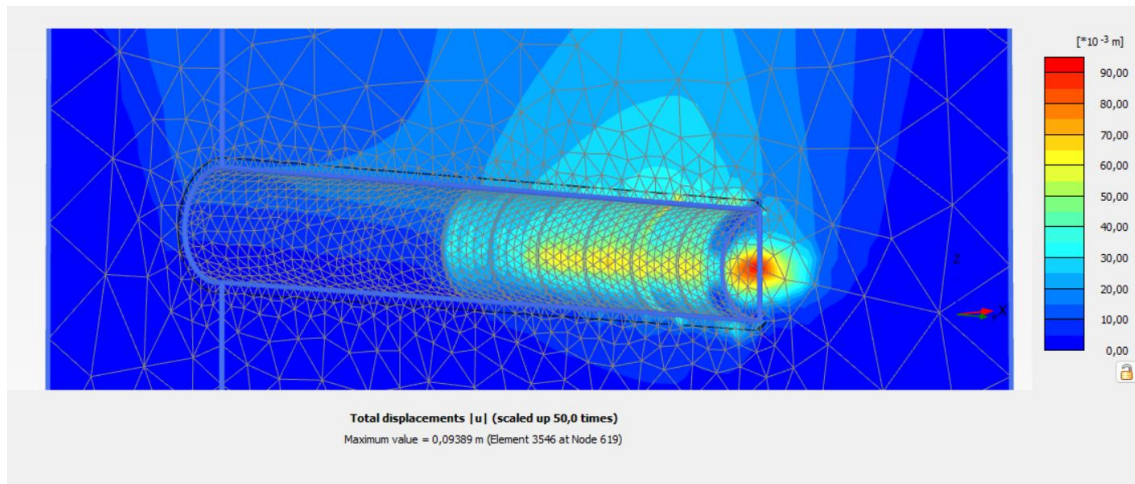


Figure 4.4 Three-dimensional finite element contour distribution of total geomechanical displacements along the active excavation drive for the deep tunneling scenario (GSI = 25, H = 139 m).

The kinematic mechanisms governing these deformation trends provide critical insights into the ground-structure interaction operating along the active mechanized drive. The pronounced expansion in radial displacements observed under low GSI conditions is directly related to the loss of rock mass shear strength and the degradation of the operational subgrade reaction modulus. In competent formations (GSI = 40), the high rock mass stiffness promotes the mobilization of an effective natural ground arch around the opening. This geological arch carries the primary share of geostatic stresses, minimizing ground convergence before the tail void gap closes. In contrast, in heavily disturbed rock masses (GSI = 20), unconfined plastic yielding propagates rapidly behind the cutterhead, substantially impairing natural arch development and transferring a larger share of the radial ground movement directly to the segmental lining.

The longitudinal attenuation of total deformations away from the active face reflects the transitional kinematics of shield-driven tunneling. In the immediate vicinity of the Fifth Ring Row, the freshly installed support elements experience the combined effects of shield tail clearance, annular gap closure, and the early consolidation of the backfill grout. During this initial stage, three-dimensional stress redistribution around the advancing shield causes maximum radial convergence. As the excavation front advances further and the distance from the face exceeds several tunnel diameters, the surrounding ground achieves static geomechanical equilibrium with the hardened grouted annulus. This stabilization arrests further displacement, establishing the asymptotic deformation plateaus observed upstream of the First Ring Row.

Furthermore, the widening displacement differentials observed between adjacent steel rings and concrete panels under deep tunneling regimes demonstrate the localized structural containment achieved by the alternating stiffness layout. Possessing an elastic modulus of 210 GPa, the structural steel rings act as rigid circumferential restraint ribs that resist radial convergence. In contrast, the relatively flexible 0.25 m concrete segments undergo controlled radial deformation between these rigid boundary points, shedding internal forces into the stiffer steel components. This composite interaction restricts unconfined tunnel ovalization under severe overburden pressures while maintaining global deformations safely within acceptable serviceability limits across all evaluated parametric scenarios.

4.2. Axial Force (N₂)

The distribution of internal lining responses across the hybrid 3+1 architecture is documented through the longitudinal variation of axial forces acting along the support elements. The parametric monitoring records the magnitude shifts and spatial profile of these compressive forces across the defined combinations of rock mass index parameters and vertical depth scales.

When analyzing the parameters of the rock mass quality, a continuous and steep upward trend in the absolute magnitudes of the axial forces is recorded as the Geological Strength Index value decreases. The complete numerical dataset capturing this geological scaling across the ring rows is detailed in Table 4.3, while the corresponding longitudinal force trajectories are plotted in Figure 4.5. Under the high-quality ground configuration where GSI equals 40, the compressive force actions along the monitored alignment remain at their lowest baseline.

However, as the geomechanical classification drops through the intermediate stages down to the weakest ground conditions at a GSI value of 20, the axial force magnitudes expand. As detailed within the data columns of Table 4.3, the peak compressive metrics more than double when tracking the transition from the highest to the lowest GSI state. Simultaneously, a distinct spatial trend is validated when tracking the force accumulation relative to the position of the ring rows behind the active TBM excavation drive. The newly placed structural components at the fifth ring row record lower compressive magnitudes compared to the older support segments located further backward along the

tunnel axis. As tracked through the row sequences of Table 4.3 and the curves in Figure 4.5, the axial force undergoes a progressive expansion when moving backward from the fresh excavation zone toward the older sequences at the second and first ring rows, before shifting toward a stabilized profile at the isolated sectors preceding the first row.

Furthermore, the continuous longitudinal monitoring reveals an intense loading contrast driven by the alternating stiffness of the support elements. As illustrated by the prominent peak-to-valley drops along the trajectories in Figure 4.5, the axial forces recorded at the specific coordinates of the high-stiffness structural steel rings are substantially larger than the force metrics tracked within the adjacent, standard precast concrete segment zones. The scale of this periodic fluctuation is heavily dependent on the GSI value. In competent ground conditions where GSI equals 40, the load distribution curve along the axis shows a more moderate variation. Conversely, as the ground quality drops to a GSI value of 20 in Table 4.3, this loading contrast expands, pushing the absolute force accumulation within the structural steel ring coordinates to its highest levels while the neighboring concrete sections carry significantly lower force boundaries.

Table 4.3 Axial Force (N_2) Values for GSI Parametric Scenarios

N_2 (kN)	S1-G20	S1-G25	S1-G30	S1-G35	S1-G40
Before First Ring Row	12800	12710	11770	10390	9626
First Ring Row	24100	20410	16340	11600	9199
Between First and Second Row	7923	6380	4968	3876	3225
Second Ring Row	27500	25110	21120	15630	11920
Between Second and Third Row	7865	6723	5529	4117	3115
Third Ring Row	22970	18770	15180	11240	7467
Between Third and Fourth Row	6736	5960	5304	4648	4077
Fourth Ring Row	19320	15850	12280	8971	6225
Between Fourth and Fifth Row	5951	5179	4638	4149	3479
Fifth Ring Row	9860	9626	8665	7269	5782

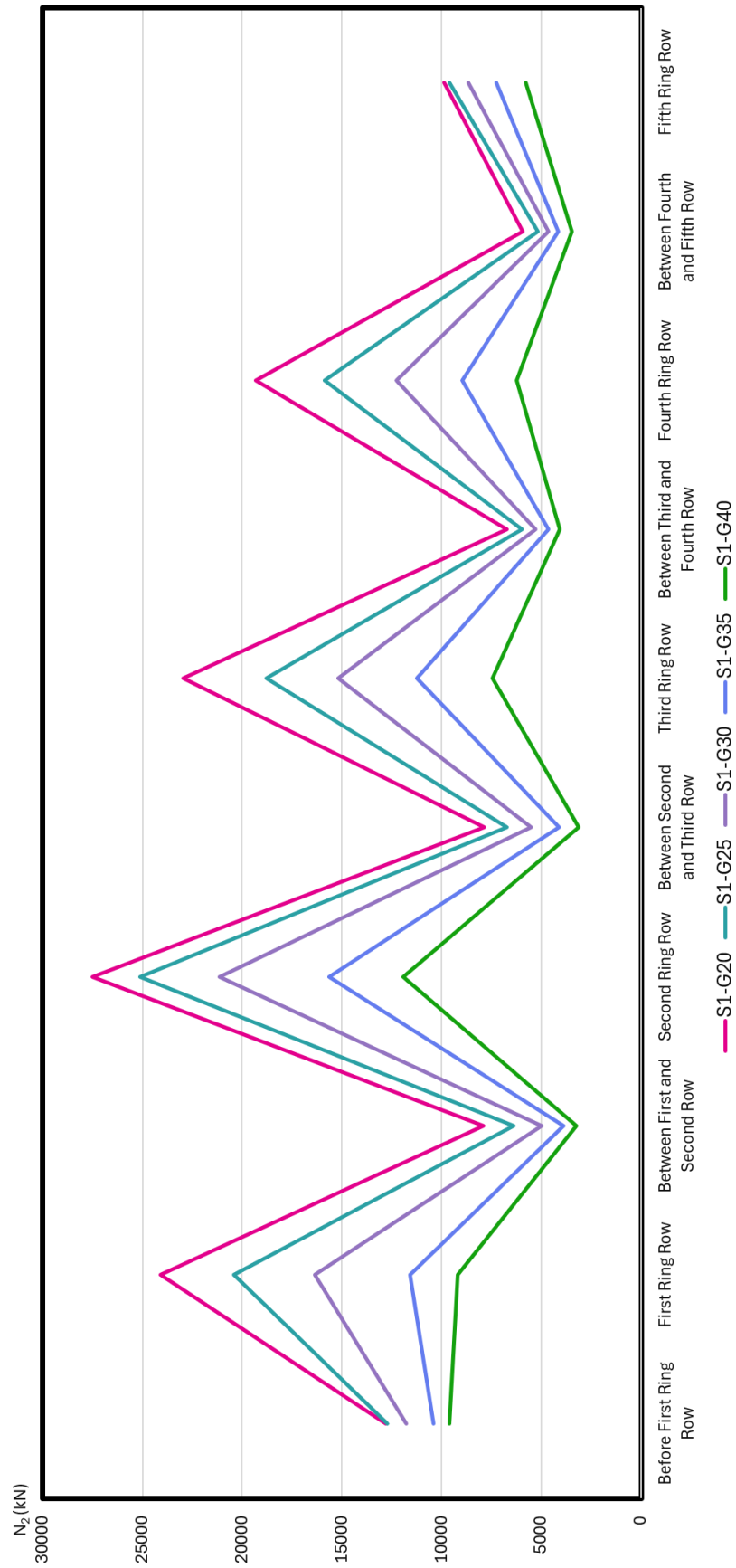


Figure 4.5 Longitudinal distribution of axial lining forces (N_2) across varying GSI configurations at a constant H of 99 m.

A parallel parametric trend is recorded when evaluating the independent scaling of the operating overburden depth. The exact numerical progression for this depth-based configuration is detailed across the stages in Table 4.4, and the resulting spatial force variations along the lining layout are plotted in Figure 4.6. As the design configuration scales upward through the five progressive depth stages from $H = 59$ m, through $H = 79$ m, $H = 99$ m, and $H = 119$ m, up to $H = 139$ m, the compressive forces experience a direct, systematic expansion across all tracking coordinates. The maximum deep configuration at $H = 139$ m drives the internal force accumulation to the absolute peak limits recorded across the entire parametric matrix, as documented in Table 4.4.

The longitudinal behavior across the ring rows remains consistent under depth scaling, showing lower force metrics at the youngest fifth ring row near the active drive and tracking higher, developed force profiles at the historical support coordinates of the second and first ring rows. However, the most critical engineering trend captured across the depth analysis involves the variation of the periodic force accumulation differences between neighboring elements. Under shallow alignment conditions characterized by $H = 59$ m in Table 4.4, the force profile along the tunnel axis tracks within a narrower boundary across both steel and concrete zones in Figure 4.6. As the overburden depth increases toward $H = 139$ m, the differences between the alternating layout components become prominent. As shown by the extreme vertical crests along the maximum depth trajectory in Figure 4.6, the high-stiffness steel rings capture severe concentrations of compressive actions under deep loading fields, whereas the neighboring concrete panels carry significantly lower force magnitudes, thereby causing the total axial force differences between neighboring structural components to widen at deeper alignment boundaries.

To fully document the cross-sectional behavior under the absolute envelope of peak structural demand across the entire parametric matrix, the envelope scenario where H equals 139 m is isolated for explicit spatial mapping. The continuous cross-sectional axial force contours are detailed in Figure 4.7, capturing the exact distribution of compressive stresses along the critical second ring row section where the absolute maximum force concentrations are recorded within the composite lining assembly.

Table 4.4 Axial Force (N_2) Values for Depth Parametric Scenarios

N_2 (kN)	S2-H59	S2-H79	S2-H99	S2-H119	S2-H139
Before First Ring Row	6574	10030	12710	15410	17470
First Ring Row	8480	14740	20410	25480	34630
Between First and Second Row	2625	4321	6380	7796	9958
Second Ring Row	9281	17200	25110	33090	39770
Between Second and Third Row	2483	4668	6723	7703	9401
Third Ring Row	5891	11140	18770	26450	33410
Between Third and Fourth Row	860	4665	5960	7187	8215
Fourth Ring Row	5203	9468	15850	22970	27940
Between Fourth and Fifth Row	2297	3577	5179	5857	7298
Fifth Ring Row	4091	7199	9626	13550	16100

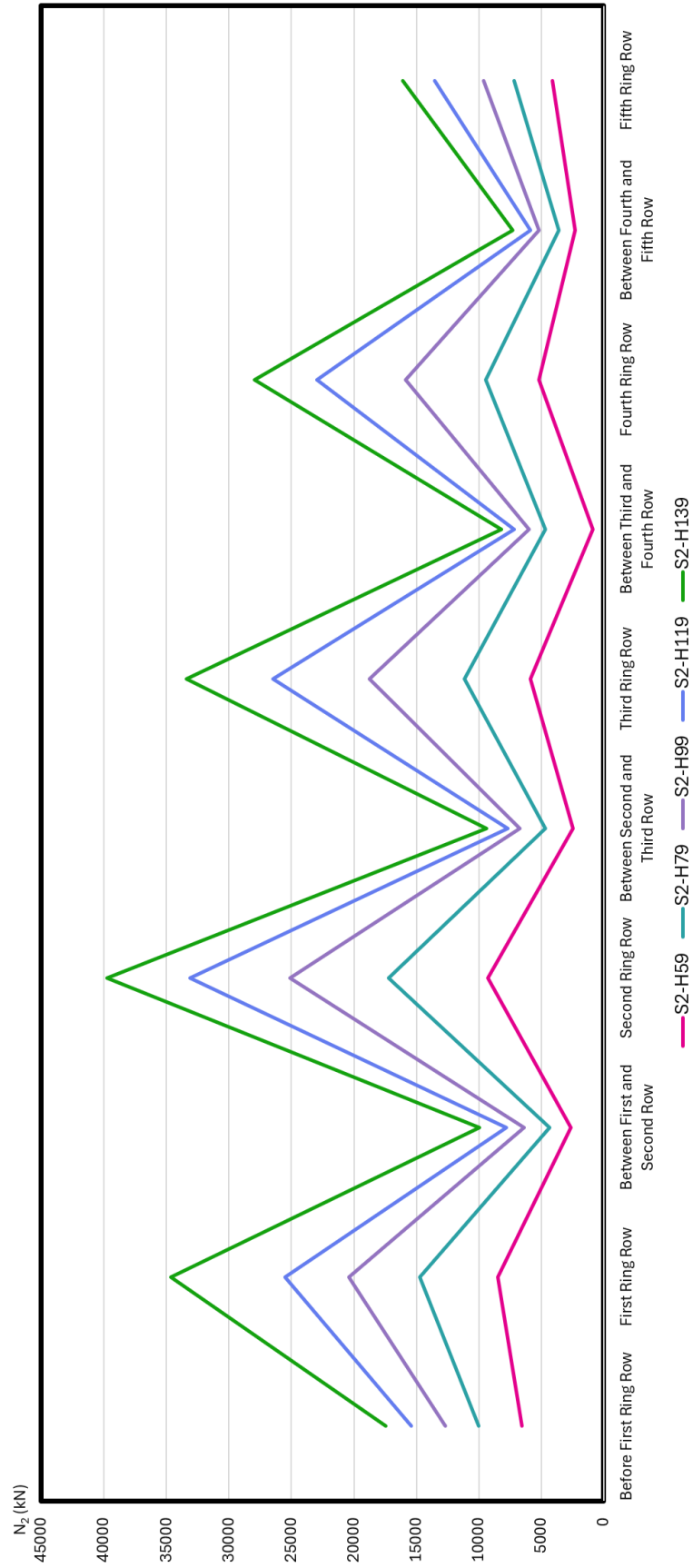


Figure 4.6 Longitudinal distribution of axial lining forces (N_2) across varying H configurations at a constant GSI of 25.

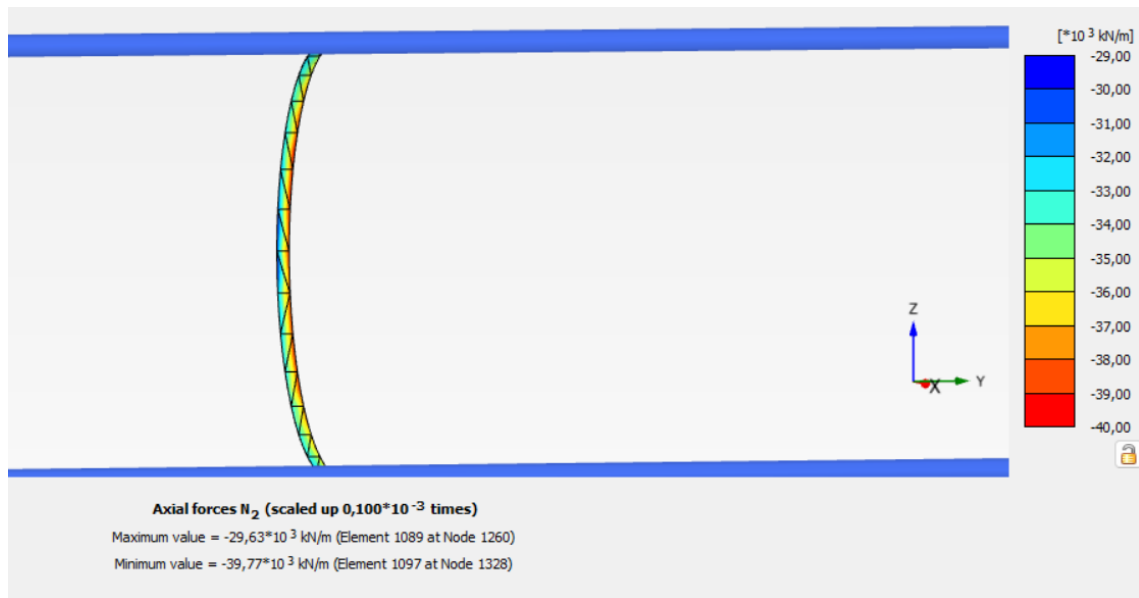


Figure 4.7 (N_2) distribution at the Second Ring Row under the peak overburden depth of $H = 139$ m.

The underlying structural and geomechanical mechanisms governing these circumferential compressive fields illustrate the active load-transfer behavior mobilized by the hybrid lining system. The pronounced escalation in normal force magnitudes under degraded rock mass conditions is primarily driven by the deterioration of shear resistance and block interlocking along geological discontinuities. In fair-to-intermediate rock masses characterized by $GSI = 40$, the intrinsic competence of the ground promotes the establishment of a stable rock arch around the excavated boundary. This natural arching mechanism carries a substantial portion of the geostatic overburden, thereby limiting the radial pressures transmitted to the lining. Conversely, when the ground competence drops to a GSI value of 20, the lack of shear strength causes extensive yielding in the plastic zone, destroying the self-supporting capacity of the ground. Under these conditions, the surrounding rock mass undergoes unconfined gravity relaxation, forcing the structural lining to directly intercept the primary lithostatic loads and leading to the observed surge in axial thrust.

The chronological accumulation of circumferential compression away from the active TBM excavation front is directly linked to the progressive closure of the annular tail void. At the newly assembled Fifth Ring Row, the support elements are situated within the partial stress shadow of the machine shield and early-age grout injection. Consequently,

the immediate geostatic demand transferred to the lining is bounded. As the machine advances further along the alignment and the distance from the face increases, the annular void closes completely, mobilizing full mechanical contact between the grouted rock boundary and the lining extrados. This transition triggers a steady, cumulative buildup of normal forces that eventually stabilizes into a steady-state equilibrium plateau upstream of the First Ring Row, indicating that long-term geostatic pressures reach an equilibrium state within the analyzed serviceability boundaries of the composite ring sequence.

Furthermore, the prominent cyclic "sawtooth" distribution observed along the longitudinal profile highlights the load-shielding efficiency generated by the structural stiffness contrast between the lining components. With an elastic modulus of 210 GPa, the structural steel rings possess significantly higher normal stiffness than the adjacent 0.25 m concrete segments (30 GPa). Because stiffer structural elements attract higher loads in indeterminate soil-structure systems, the internal steel rings actively intercept a disproportionate share of the circumferential thrust. This mechanism relieves the neighboring concrete segments from extreme compressive stress concentrations, keeping concrete stress levels within an optimal, uniform compressive range and preventing premature crushing or buckling along the segment joints. The cross-sectional distribution mapped in Figure 4.7 further demonstrates that hoop thrust concentrates predominantly along the tunnel sidewalls (springlines), confirming that the continuous lateral geostatic confinement is efficiently mobilized by the hybrid system under deep overburden conditions.

4.3. Shear Force (Q_{12})

The structural demand profiles acting upon the alternating lining assembly are documented through the distribution of in-plane shear forces (Q_{12}) developed along the successive ring sequences. The parametric configurations record the magnitude variations and longitudinal profiles of these shear actions across the specified rock mass conditions and vertical overburden depths.

When tracking the influence of the rock mass quality parameters, a continuous and distinct upward trend in the absolute shear force magnitudes is captured as the Geological Strength Index value decreases. The complete numerical progression documenting this geomechanical scaling across all examined states is detailed in Table 4.5, while the

corresponding longitudinal shear force trajectories are plotted in Figure 4.8. In the ground configuration characterized by a high GSI value of 40, the internal shear force actions remain well-controlled and track along the lowest baseline metrics.

However, as the ground quality progressively drops through the intermediate classifications down to the poorest rock mass environment at a GSI value of 20, the shear force magnitudes undergo a direct expansion. As detailed across the data columns of Table 4.5, the peak shear metrics recorded along the lining sections exhibit a substantial increase when moving from the highest GSI baseline to the lowest GSI classification stage.

Simultaneously, a systematic spatial trend is validated when monitoring the force distribution relative to the position of the structural components behind the active TBM excavation drive. The youngest structural components located closest to the active face, specifically at the fifth ring row position, record lower shear magnitudes compared to the older support segments located further backward along the tunnel axis. As tracked through the row sequences of Table 4.5 and the wave-like curves in Figure 4.8, the shear forces experience a progressive expansion when moving backward from the active excavation zone toward the older support sequences at the second and first ring rows, before settling into a stabilized profile at the stable historical sectors preceding the first row.

Furthermore, the continuous longitudinal tracking reveals an intense shear force contrast driven by the alternating stiffness of the composite lining components. As shown by the prominent peak-to-valley drops along the trajectories in Figure 4.8, the shear forces recorded at the specific coordinates of the high-stiffness structural steel rings are substantially larger than the force metrics tracked within the adjacent, standard precast concrete segment zones. The intensity of this periodic fluctuation is heavily dependent on the surrounding GSI parameter. In competent ground conditions where GSI equals 40, the shear force profile along the tunnel alignment tracks within a narrower, more uniform band. Conversely, as the ground quality drops to a GSI value of 20 in Table 4.5, this shear loading contrast expands significantly, causing extreme force concentrations at the structural steel ring coordinates while the neighboring concrete sections remain at lower shear boundaries).

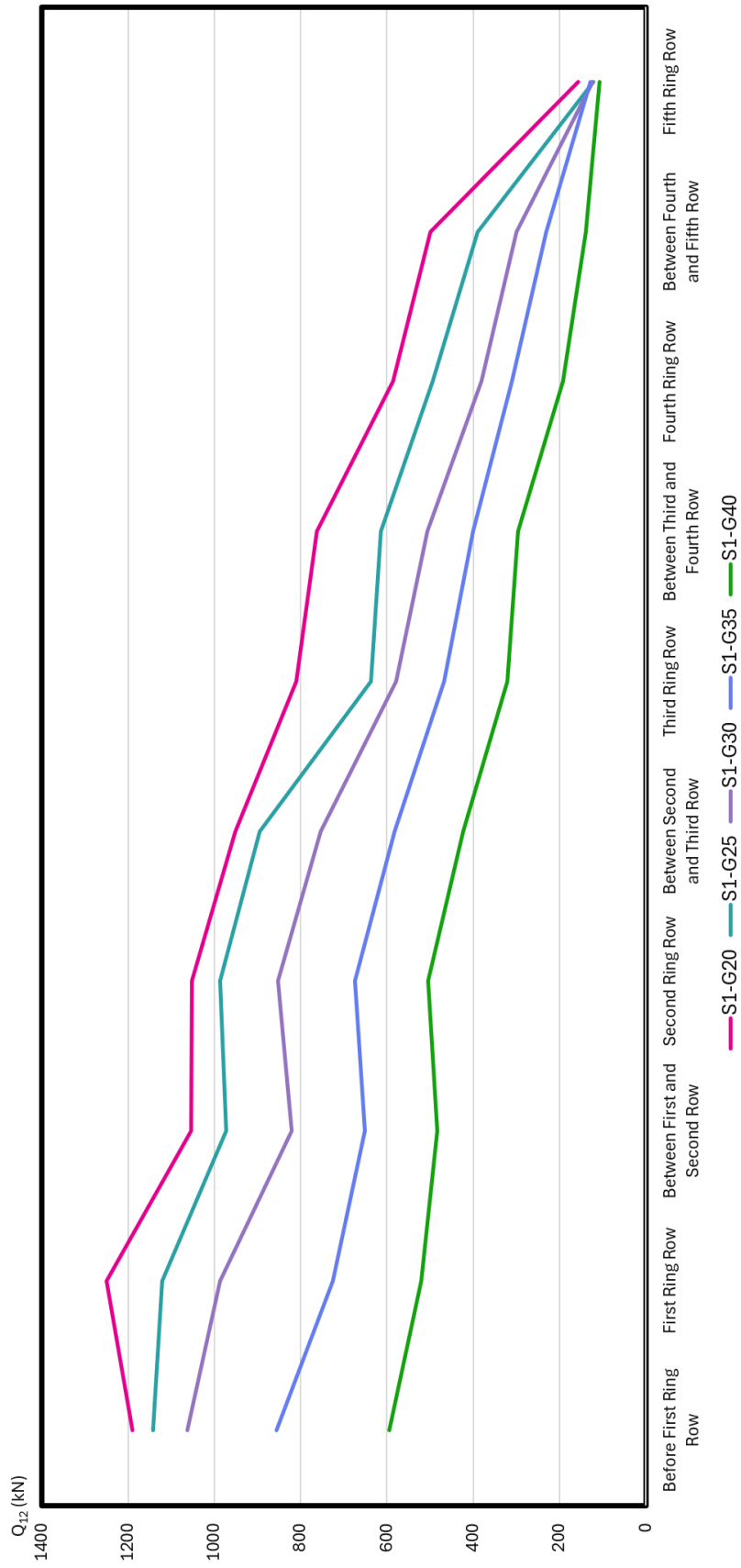


Figure 4.8 Longitudinal distribution of maximum cross-sectional shear forces (Q_{12}) across varying GSI configurations at constant H of 99 m.

Table 4.5 Shear Force (Q_{12}) Values for GSI Parametric Scenarios

Q_{12} (kN)	S1-G20	S1-G25	S1-G30	S1-G35	S1-G40
Before First Ring Row	1190	1141	1062	855	595
First Ring Row	1249	1121	987	725	521
Between First and Second Row	1054	973	820	651	484
Second Ring Row	1051	986	853	674	505
Between Second and Third Row	951	895	754	583	423
Third Ring Row	809	637	578	468	321
Between Third and Fourth Row	763	614	506	401	297
Fourth Ring Row	585	493	381	310	192
Between Fourth and Fifth Row	499	390	299	231	139
Fifth Ring Row	156	121	123	129	108

A parallel parametric trend is recorded when evaluating the independent scaling of the operating overburden depth parameter. The exact numerical progression for this depth-based loading configuration is detailed across the stages in Table 4.6, and the resulting spatial shear force variations along the lining layout are plotted in Figure 4.9. As the design configuration scales upward through the five progressive depth stages from $H = 59$ m, through $H = 79$ m, $H = 99$ m, and $H = 119$ m, up to the maximum deep environment of $H = 139$ m, the induced shear forces undergo a direct, systematic expansion across all tracking coordinates. The maximum deep configuration at $H = 139$ m drives the internal shear force accumulation to the absolute peak limits recorded across the entire parametric matrix, as documented in Table 4.6.

The longitudinal behavior across the ring rows remains consistent under depth scaling, showing lower shear metrics at the youngest fifth ring row near the active drive and tracking higher, developed shear force profiles at the historical support coordinates of the second and first ring rows. However, the variation of the periodic shear differences between neighboring elements shows distinct profiles across the depth stages. Under shallow alignment conditions characterized by $H = 59$ m in Table 4.6, the force profile along the tunnel axis tracks within a narrower boundary across both steel and concrete zones in Figure 4.9, showing that the structural shear values remain close to each other along the longitudinal alignment.

As the overburden depth increases toward $H = 139$ m, the differences between the alternating layout components become prominent. As shown by the severe peak-to-valley drops along the maximum depth trajectory in Figure 4.9, the high-stiffness steel rings capture severe concentrations of shear actions under deep loading fields, whereas the neighboring concrete panels carry significantly lower force magnitudes, thereby causing the total shear force differences between neighboring structural components to widen at deeper alignment boundaries.

To fully document the cross-sectional behavior under the absolute envelope of peak structural demand across the entire parametric matrix, the envelope scenario where H equals 139 m is isolated for explicit spatial mapping. The continuous cross-sectional shear force contours are detailed in Figure 4.10, capturing the exact distribution of in-plane shear stresses along the critical first ring row section where the absolute maximum force concentrations are recorded within the composite lining assembly.

Table 4.6 Shear Force (Q_{12}) Values for Depth Parametric Scenarios

Q_{12} (kN)	S2-H59	S2-H79	S2-H99	S2-H119	S2-H139
Before First Ring Row	685	994	1141	1220	1506
First Ring Row	514	886	1121	1323	1527
Between First and Second Row	450	761	973	1184	1373
Second Ring Row	434	713	986	1128	1284
Between Second and Third Row	394	710	895	1035	1168
Third Ring Row	253	444	637	967	1074
Between Third and Fourth Row	253	441	614	881	1036
Fourth Ring Row	147	279	493	681	828
Between Fourth and Fifth Row	111	210	390	567	789
Fifth Ring Row	42	92	121	219	257

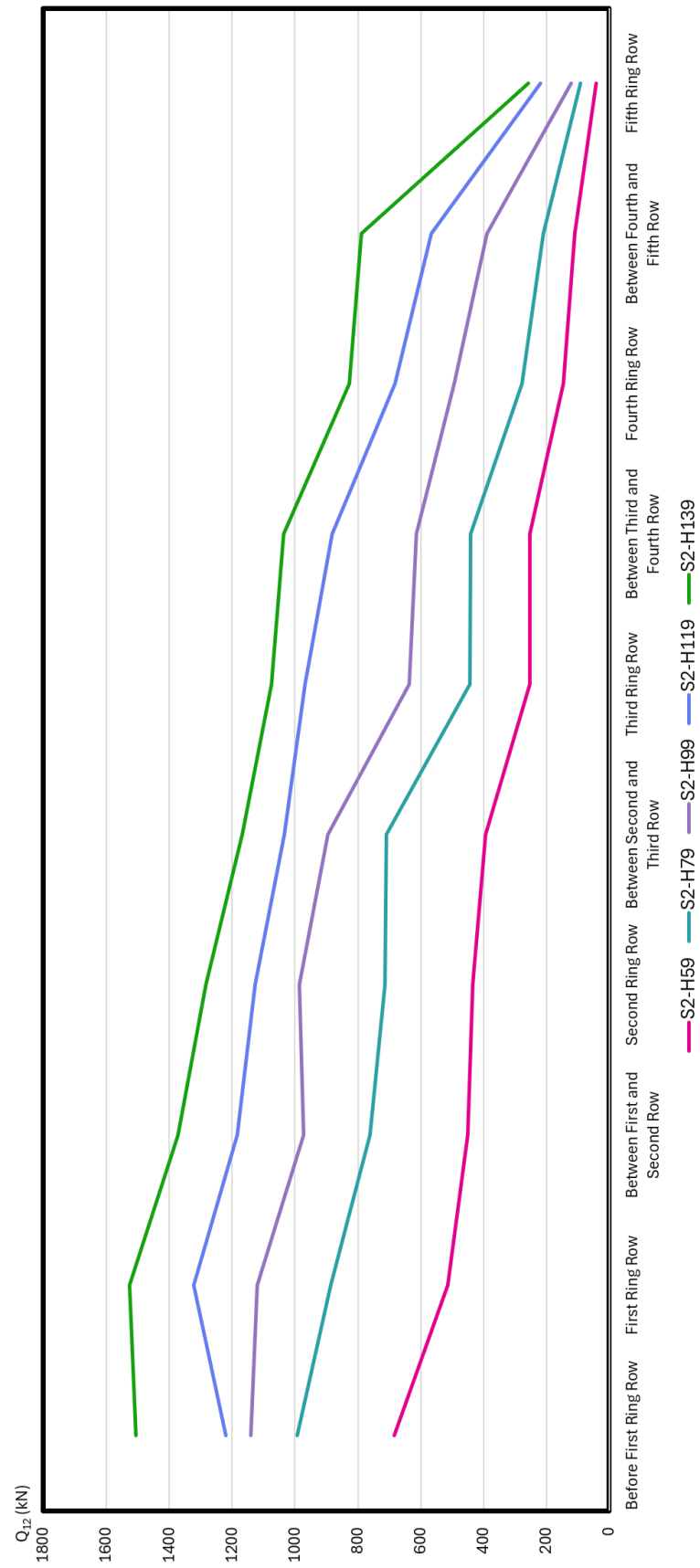


Figure 4.9 Longitudinal distribution of maximum cross-sectional shear forces (Q_{12}) across varying H configurations at a constant GSI of 25.

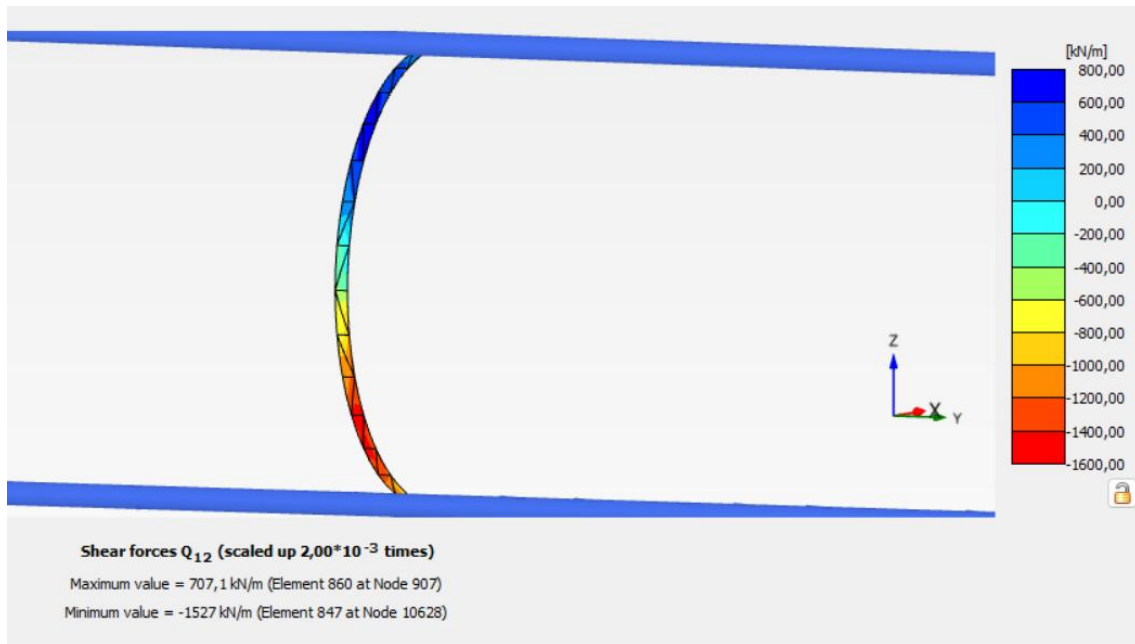


Figure 4.10 In-plane shear force (Q_{12}) distribution at the First Ring Row under the peak overburden depth of $H = 139$ m.

The mechanics governing the development and distribution of in-plane shear forces (Q_{12}) provide critical understanding of how the alternating lining configuration accommodates asymmetric ground loads and preserves cross-sectional equilibrium. The continuous escalation in shear force magnitudes observed under degrading rock mass conditions is directly attributed to the non-uniform shear stress fields mobilized within the surrounding plastic zone. In competent rock masses ($GSI = 40$), the high shear strength and deformation modulus of the ground limit asymmetric ground relaxation, which keeps the resulting transverse shear demands well-contained across all monitoring sections. However, as the rock quality deteriorates toward $GSI = 20$, the ground experiences progressive yielding and localized shear band development along discontinuities. This creates uneven radial pressures along the tunnel perimeter, compelling the support system to mobilize high internal shear resistances to maintain structural circularity and prevent ring distortion.

The spatial distribution along the longitudinal axis reflects the progressive transfer of transverse ground stresses behind the advancing shield. At the newly erected Fifth Ring Row, the structural components are partially shielded by the rigid TBM skin and the fresh backfill grout layer, resulting in lower early-age shear demands. As the excavation advances forward, the annular gap closes completely and the surrounding ground achieves intimate contact with the hardened grouted support. This transition induces a

cumulative rise in transverse shear actions that stabilizes into a steady-state profile toward the historical support rows, verifying that long-term asymmetric geostatic loads are safely equilibrated along the drive.

Crucially, the cross-sectional orientation of the peak shear forces and the periodic fluctuations between neighboring support elements highlight the structural efficiency of the hybrid layout. As demonstrated by the cross-sectional contour mapping in Figure 4.10, in-plane shear forces consistently concentrate symmetrically along the vertical axis at the tunnel crown and invert. Under increasing overburden depths, the vertical gravitational overburden combined with radial relaxation generates severe transverse shearing actions at these two antipodal zones. Due to their high in-plane shear rigidity, the structural steel rings act as stiff internal load-bearing diaphragms that actively intercept and absorb these vertical shear concentrations. This load-transfer mechanism significantly reduces the transverse shear demands transferred to the adjacent 0.25 m concrete segments, thereby limiting potential joint distress, reducing the likelihood of radial dislocations, and supporting the structural stability of the thin-walled configuration under the simulated conditions.

4.4. Bending Moment (M_{11})

The structural flexural behavior of the composite 3+1 segmental tunnel lining system is evaluated through the distribution and magnitude shifts of the maximum in-plane bending moments (M_{11}) developing across sequential support rings along the longitudinal advancement axis. The parametric configurations record the magnitude variations and longitudinal profiles of these bending moments across the specified rock mass conditions and vertical overburden depths.

When tracking the influence of the rock mass quality parameters, a continuous and distinct downward trend in the absolute bending moment magnitudes is captured as the Geological Strength Index value increases. The entire numerical progression of this geomechanical scaling over all the states is presented in Table 4.7, whereas the corresponding longitudinal bending moment trajectories are plotted in Figure 4.11. In the ground configuration characterized by a high GSI value of 40, the internal bending moments remain well-controlled and track along the lowest baseline metrics.

However, as the ground quality progressively drops through the intermediate classifications down to the poorest rock mass environment at a GSI value of 20, the bending moment magnitudes undergo a direct expansion. As detailed across the data columns of Table 4.7, the peak bending metrics recorded along the lining sections exhibit a substantial increase when moving from the highest GSI baseline to the lowest GSI classification stage.

Simultaneously, a systematic spatial trend is validated when monitoring the moment distribution relative to the position of the structural components behind the active TBM excavation drive. The youngest structural components located closest to the active face, specifically at the fifth ring row position, record lower bending magnitudes compared to the older support segments located further backward along the tunnel axis. As tracked through the row sequences of Table 4.7 and the curves in Figure 4.11, the bending moments experience a progressive expansion when moving backward from the active excavation zone toward the older support sequences at the second and first ring rows, before settling into a stabilized profile at the stable historical sectors preceding the first row.

Furthermore, the continuous longitudinal tracking reveals an intense bending moment contrast driven by the alternating stiffness of the composite lining components. As shown by the prominent peak-to-valley drops along the trajectories in Figure 4.11, the bending moments recorded at the specific coordinates of the high-stiffness structural steel rings are substantially larger than the moment metrics tracked within the adjacent, standard precast concrete segment zones. The intensity of this periodic fluctuation is heavily dependent on the surrounding GSI parameter. In competent ground conditions where GSI equals 40, the bending moment profile along the tunnel alignment tracks within a narrower, more uniform band. Conversely, as the ground quality drops to a GSI value of 20 in Table 4.7, this flexural loading contrast expands significantly, causing extreme moment concentrations at the structural steel ring coordinates while the neighboring concrete sections remain at lower bending boundaries.

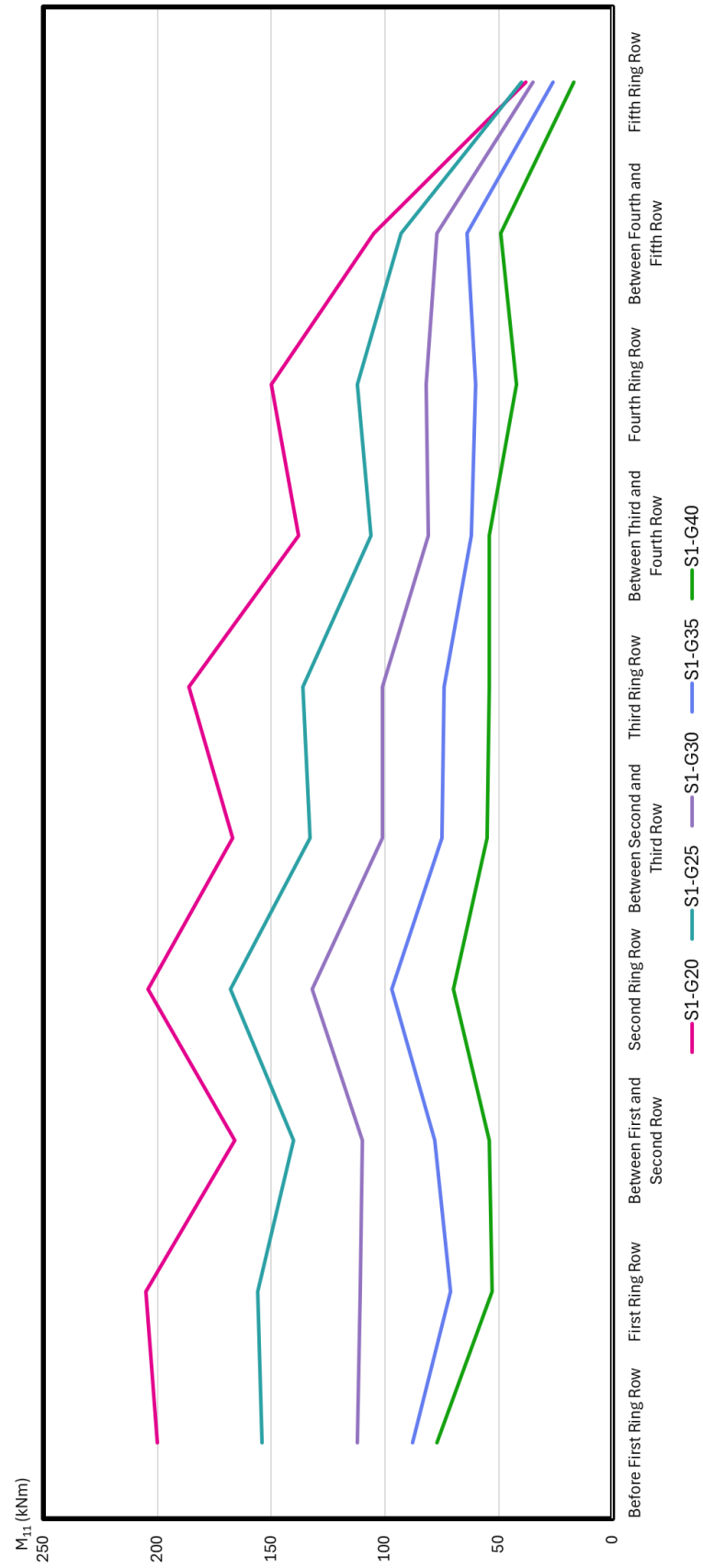


Figure 4.11 Longitudinal distribution of maximum cross-sectional bending moments (M_{11}) across varying GSI configurations at constant H of 99 m.

Table 4.7 Bending Moment (M_{11}) Values for GSI Parametric Scenarios

M_{11} (kNm)	S1-G20	S1-G25	S1-G30	S1-G35	S1-G40
Before First Ring Row	200	154	112	88	77
First Ring Row	205	156	111	71	53
Between First and Second Row	166	140	110	78	54
Second Ring Row	204	168	132	97	70
Between Second and Third Row	167	133	101	75	55
Third Ring Row	186	136	101	74	54
Between Third and Fourth Row	138	106	81	62	54
Fourth Ring Row	150	112	82	60	42
Between Fourth and Fifth Row	105	93	77	64	49
Fifth Ring Row	38	40	35	26	17

A parallel parametric trend is recorded when evaluating the independent scaling of the operating overburden depth parameter. The exact numerical progression for this depth-based loading configuration is detailed across the stages in Table 4.8, and the resulting spatial bending moment variations along the lining layout are plotted in Figure 4.12. As the design configuration scales upward through the five progressive depth stages from $H = 59$ m, through $H = 79$ m, $H = 99$ m, and $H = 119$ m, up to the maximum deep environment of $H = 139$ m, the induced bending moments undergo a direct, systematic expansion across all tracking coordinates. The maximum deep configuration at $H = 139$ m drives the internal bending moment accumulation to the absolute peak limits recorded across the entire parametric matrix, as documented in Table 4.8.

The longitudinal behavior across the ring rows remains consistent under depth scaling, showing lower bending metrics at the youngest fifth ring row near the active drive and tracking higher, developed bending moment profiles at the historical support coordinates of the second and first ring rows. However, the variation of the periodic bending differences between neighboring elements shows distinct profiles across the depth stages. Under shallow alignment conditions characterized by $H = 59$ m in Table 4.8, the force profile along the tunnel axis tracks within a narrower boundary across both steel and concrete zones in Figure 4.12, showing that the structural bending values remain close to each other along the longitudinal alignment.

As the overburden depth increases toward $H = 139$ m, the differences between the alternating layout components become prominent. As shown by the severe peak-to-valley drops along the maximum depth trajectory in Figure 4.12, the high-stiffness steel rings capture severe concentrations of bending moments under deep loading fields, whereas the neighboring concrete panels carry significantly lower moment magnitudes, thereby causing the total bending moment differences between neighboring structural components to widen at deeper alignment boundaries.

Table 4.8 Bending Moment (M_{11}) Values for Depth Parametric Scenarios

M_{11} (kNm)	S2-H59	S2-H79	S2-H99	S2-H119	S2-H139
Before First Ring Row	57	108	154	202	282
First Ring Row	51	111	156	213	298
Between First and Second Row	45	96	140	169	223
Second Ring Row	55	119	168	226	309
Between Second and Third Row	42	91	133	146	215
Third Ring Row	44	77	136	186	252
Between Third and Fourth Row	36	68	106	148	182
Fourth Ring Row	37	67	112	170	204
Between Fourth and Fifth Row	33	58	93	105	130
Fifth Ring Row	12	28	39	63	69

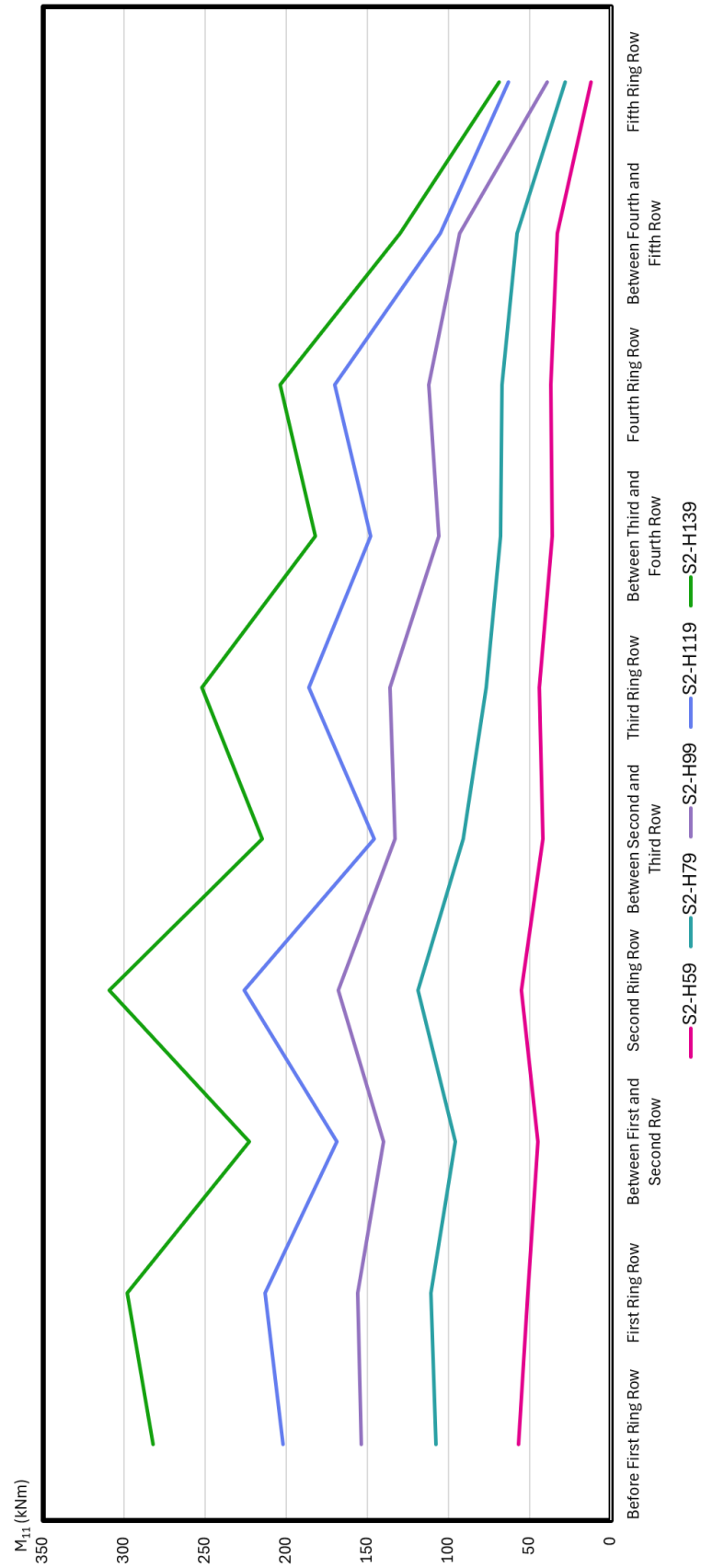


Figure 4.12 Longitudinal distribution of maximum cross-sectional bending moments (M_{11}) across varying H configurations at a constant GSI of 25.

To fully document the cross-sectional behavior under the absolute envelope of peak structural demand across the entire parametric matrix, the envelope scenario where H equals 139 m is isolated for explicit spatial mapping. The continuous cross-sectional bending moment contours are detailed in Figure 4.13, capturing the exact distribution of in-plane bending stresses along the critical second ring row section where the absolute maximum force concentrations are recorded within the composite lining assembly.



Figure 4.13 In-plane bending moment (M_{11}) distribution at the Second Ring Row under the peak overburden depth of $H = 139$ m.

The flexural behavior and moment redistribution mechanisms observed across the hybrid lining assembly provide fundamental insights into how the alternating stiffness configuration counteracts cross-sectional distortion and mitigates eccentric stress concentrations. The pronounced reduction in bending moment magnitudes under increasing GSI values demonstrates the governing role of the rock mass self-supporting capacity. In fair-to-intermediate rock masses ($GSI = 40$), the high shear strength and tight block interlocking promote the formation of an effective natural ground arch around the excavation cavity. This geological arch carries the primary share of the non-uniform geostatic pressure, resulting in a more isotropic radial stress field acting on the lining extrados and consequently reducing in-plane flexural actions across all sections. Conversely, when the rock mass quality deteriorates to $GSI = 20$, the absence of stable ground arching triggers asymmetric plastic yielding and localized stress concentrations

around the cavity perimeter. The resulting non-uniform radial loading imposes severe flexural demands on the lining, causing the sharp escalation in bending moments observed across the low-GSI models.

The longitudinal evolution of in-plane bending moments away from the active TBM front reflects the staged transfer of geostatic confinement behind the advancing shield. At the newly erected Fifth Ring Row, bending actions remain minimal because the segments are partially shielded by the rigid tail skin of the machine and the liquid backfill grout layer. As the TBM advances along the alignment, the tail void gap closes, and the surrounding rock mass establishes direct mechanical contact with the hardened grouted lining. This progressive interaction mobilizes continuous cross-sectional ovalization tendencies, driving a steady accumulation of flexural moments until a stable, steady-state moment plateau is achieved upstream of the First Ring Row.

Furthermore, the pronounced cyclic "sawtooth" undulation visible along the longitudinal moment trajectories highlight the structural shielding provided by the internal steel rings. Because the structural steel rings possess a flexural rigidity significantly higher than that of the adjacent 0.25 m concrete segments, they act as rigid structural stiffeners that actively attract and isolate the peak bending distortions. This deliberate stiffness contrast establishes a mechanical filter: the high-ductility steel rings absorb the localized moment peaks, thereby relieving the intermediate concrete panels from severe flexural demands. As demonstrated by the cross-sectional contour mapping in Figure 4.13, maximum in-plane bending moments concentrate along the lateral sidewalls (springlines) under high overburden depths. By intercepting these sidewall flexural concentrations within the steel rings, the hybrid layout promotes a predominantly compressive stress regime in the adjacent concrete panels with reduced eccentricity, which helps mitigate tensile cracking risks and maintains structural serviceability under the examined geostatic environments.

4.5. Evaluation of Other Structural Forces on Lining (N_1 , M_{22} , M_{12} , Q_{23} , Q_{13})

To establish a comprehensive and structurally rigorous verification of the proposed hybrid support configuration, it is computationally imperative to evaluate the secondary internal structural forces generated within the lining assembly under complex three-dimensional geomechanical stress fields. While the primary design criteria in tunnel engineering are traditionally governed by structural convergence (u), circumferential

hoop thrust (N_2), in-plane bending moments (M_{11}), and primary shear forces (Q_{12}), the continuous interaction between the rock mass continuum and the sequential TBM excavation triggers multi-axial loading states that cannot be captured by conventional two-dimensional analytical frameworks.

Therefore, this section systematically cross-examines the distribution and parametric sensitivity of the remaining internal forces provided by the PLAXIS 3D post-processing domain, specifically focusing on the longitudinal axial forces (N_1), out-of-plane longitudinal bending moments (M_{22}), torsional moments (M_{12}), and out-of-plane shear forces (Q_{13} and Q_{23}). Given the highly optimized structural stiffness of the alternating 25 cm composite lining arrangement, these secondary operational demands are evaluated collectively to ensure that eccentric loading combinations, out-of-plane warping tendencies, and localized cross-sectional shear distributions remain strictly within safe ultimate limit state (ULS) design thresholds across all evaluated Geological Strength Index (GSI) domains and overburden depths (H).

4.5.1. Longitudinal Axial Force and Bending (N_1 and M_{22})

The longitudinal performance and axial load distribution of the composite support sequence are evaluated by analyzing the maximum longitudinal axial forces (N_1) and out-of-plane longitudinal bending moments (M_{22}) across sequential lining rings. Figure 4.14 and Figure 4.15 illustrate the longitudinal distribution of the maximum axial forces (N_1) across varying Geological Strength Index (GSI) regimes and overburden depth (H) steps, respectively.

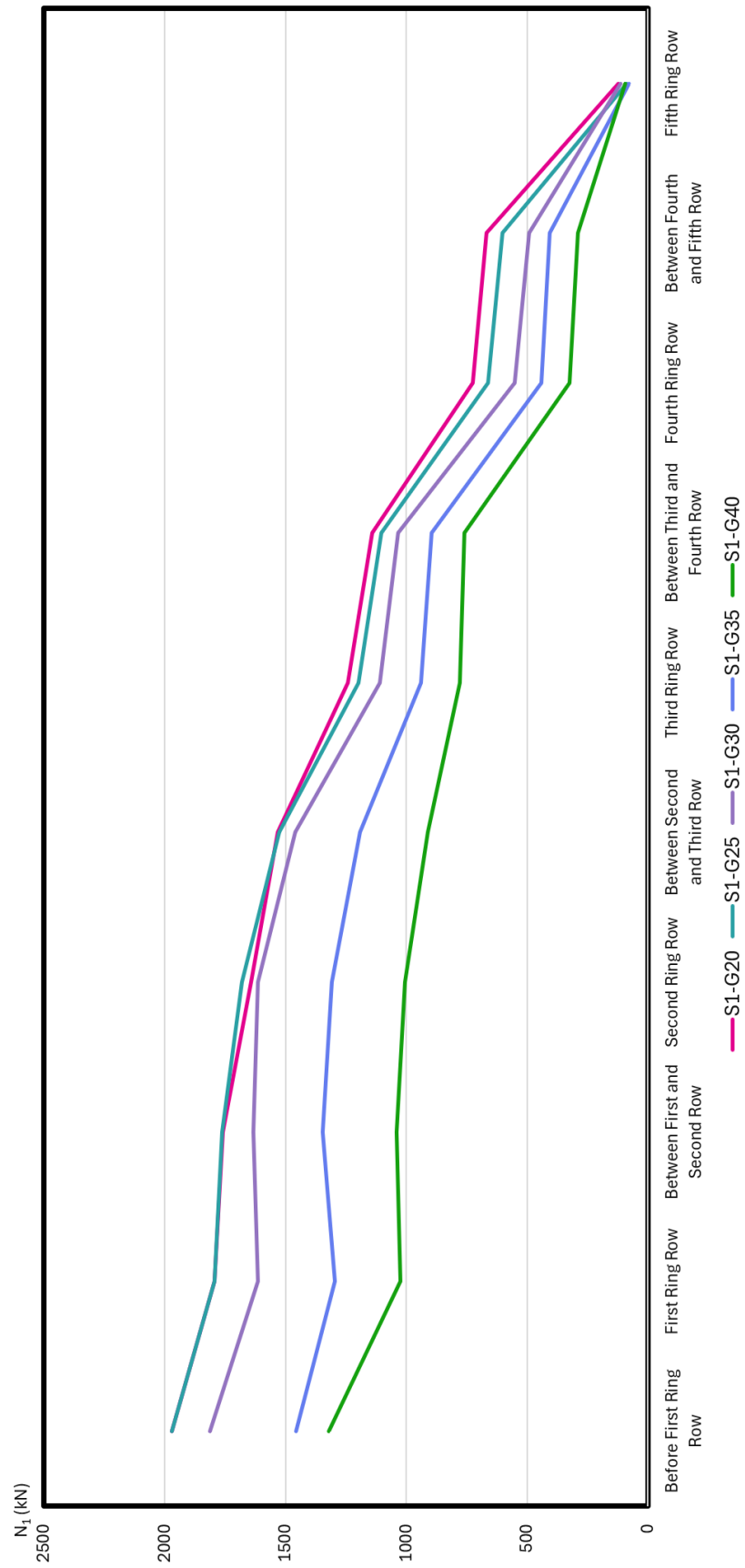


Figure 4.14 Longitudinal distribution of maximum cross-sectional longitudinal axial forces (N_1) across varying GSI configurations at a H of 99 m.

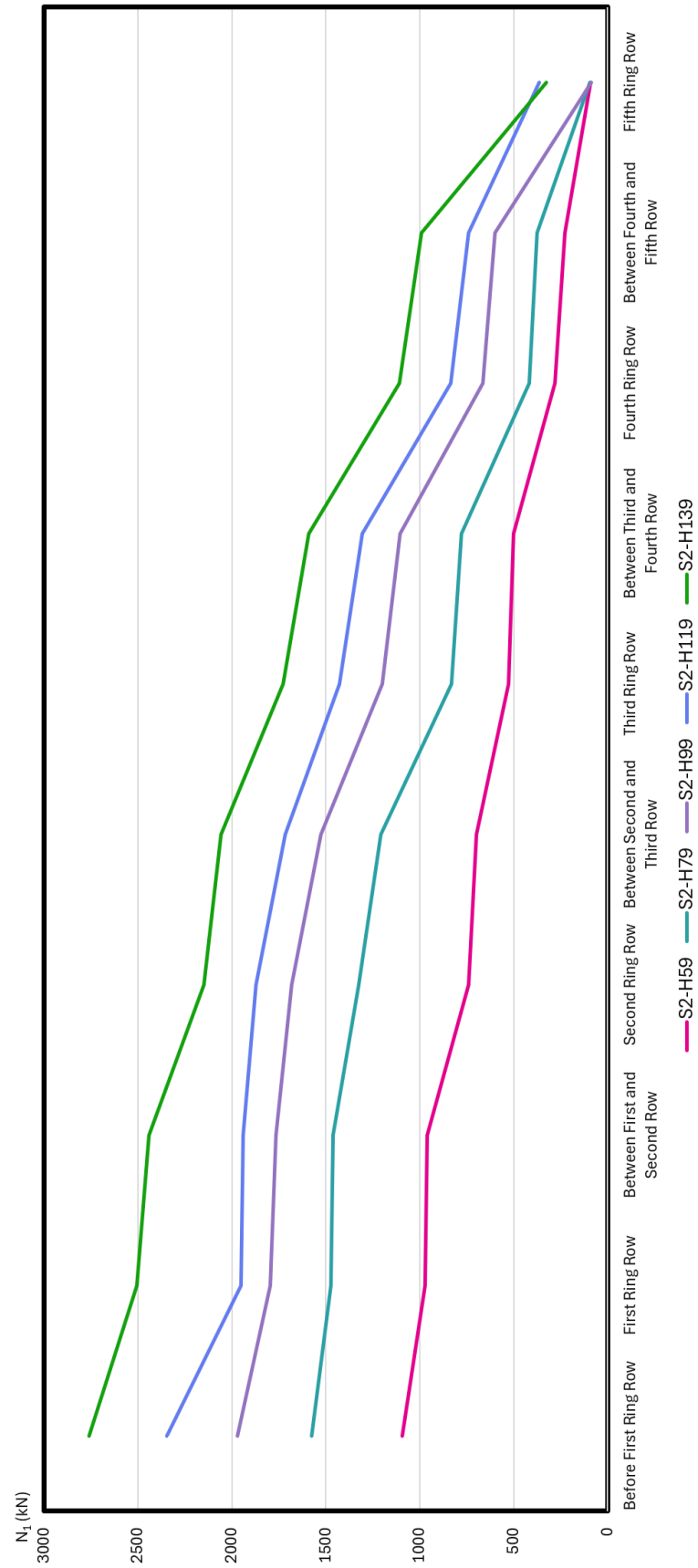


Figure 4.15 Longitudinal distribution of maximum cross-sectional longitudinal forces (N_1) across varying H configurations at a constant GSI of 25.

A macroscopic evaluation of the numerical output indicates that the longitudinal axial force (N_1) accumulation is highly sensitive to both the geomechanical competence of the rock mass and the total operating overburden pressure. As the ground quality transitions from highly degraded to intermediate regimes (GSI=20 to GSI=40), a systematic reduction in maximum N_1 forces is captured from approximately 1980 kN to 1320 kN. Conversely, scaling the depth configuration from $H=59$ m to $H=139$ m induces a severe, proportional expansion in the longitudinal loading fields, escalating the maximum structural force from a baseline value of 1100 kN to a substantial peak of 2760 kN, representing an increase of over 150%.

A critical spatial trend is captured when monitoring the N_1 profiles in relation to the distance from the advanced TBM excavation profile. In the sections closest to the active drive zone ("Fifth Ring Row"), the induced longitudinal forces drop toward their absolute minimum limits across all evaluated simulation series. This transient relaxation profile materializes because the freshly installed support rings are structurally isolated from the long-term longitudinal stress state during early-stage shield tail clearance. However, as the excavation alignment progresses further along the negative Y-axis towards the older, stabilized tunnel zone ("Before First Ring Row"), the sequential lining rings achieve a fully engaged state of mechanical equilibrium with the surrounding continuous rock formation. This structural transition causes a steady, cumulative accumulation of longitudinal thrust before arriving at its ultimate steady-state plateau at the far end of the monitoring trajectory.

To supplement the axial force tracking, the induced out-of-plane longitudinal bending moments (M_{22}) developing simultaneously across the advanced support geometry are systematically plotted in Figure 4.16 and Figure 4.17.

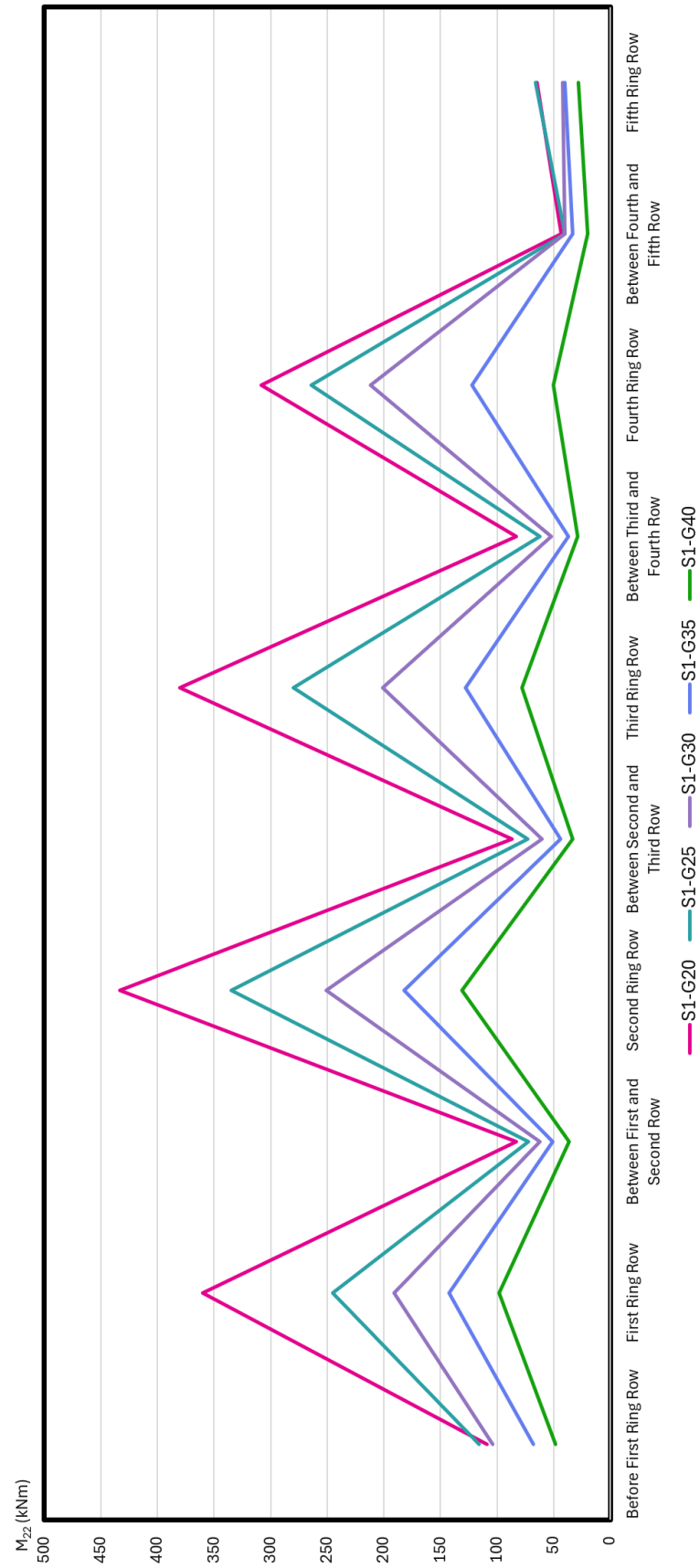


Figure 4.16 Longitudinal distribution of maximum cross-sectional longitudinal bending moments (M_{22}) across varying GSI configurations at a constant H of 99 m.

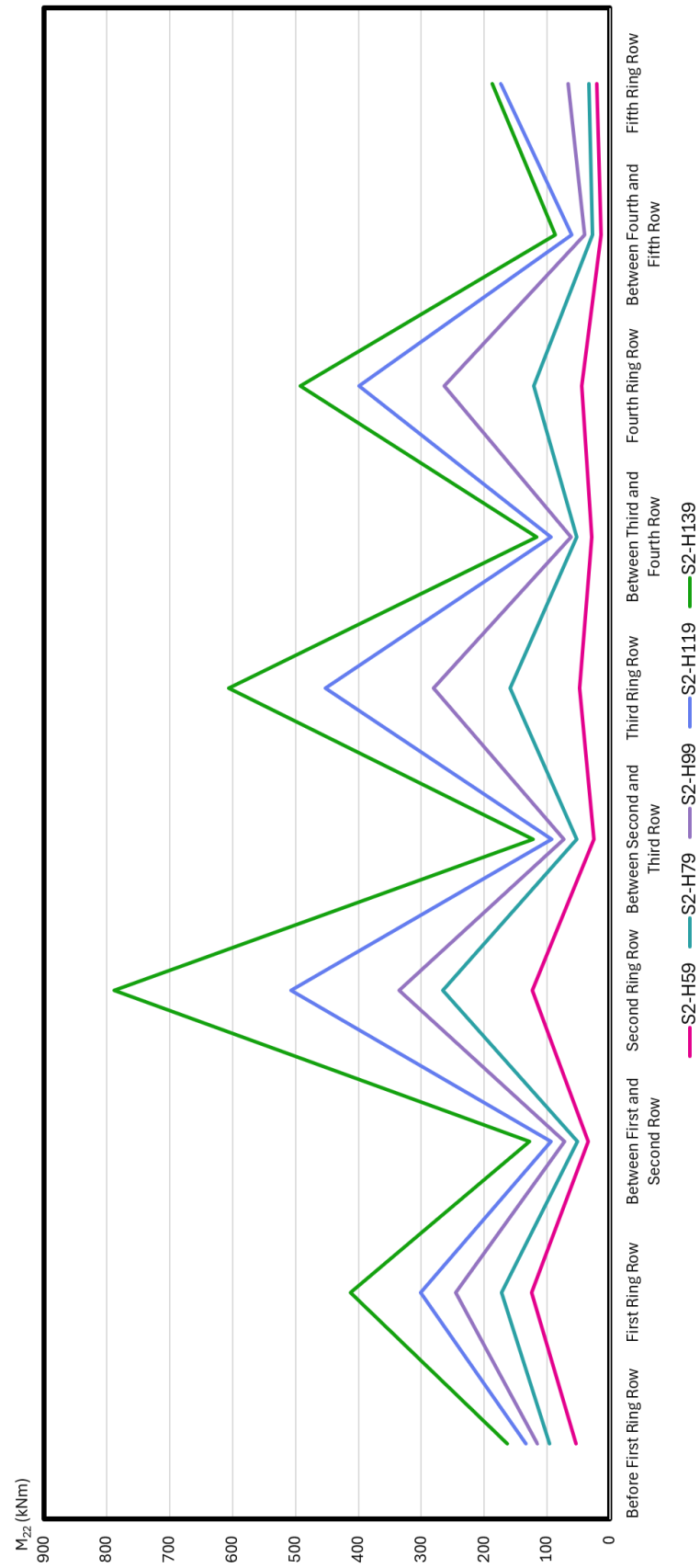


Figure 4.17 Longitudinal distribution of maximum cross-sectional longitudinal bending moments (M_{22}) across varying H configurations at a constant GSI of 25.

Crucially, the spatial distribution of the longitudinal bending moments (M_{22}) exhibits an aggressive, cyclic "sawtooth" fluctuation pattern that remains prominently visible across all evaluated ground qualities and tunneling depths. This repeating mechanical trend is directly governed by the distinct cross-sectional stiffness contrast and the non-continuous structural connectivity inherent to the 3+1 alternating support architecture. The local minima correspond precisely to the core structural matrices of the standard concrete configurations. As the longitudinal trajectory tracks across the ring interfaces where the high-stiffness structural steel rings are activated, the localized structural constraints trigger rapid moment transfers, driving sharp, periodic peaks along the alignment.

The structural sensitivity of these localized M_{22} peaks is strongly amplified under poor geomechanical conditions and deep tunneling environments. Under highly degraded rock mass regimes ($GSI=20$), the maximum localized bending moment reaches a significant peak value of 435 kNm, whereas the value is strictly constrained below 132 kNm under enhanced ground conditions ($GSI=40$). Furthermore, as the overburden depth scales to the deep configuration of $H=139$ m, the heightened geostatic pressures transferred to the lining assembly expand the structural zig-zag moment deflection up to a maximum peak of 790 kNm, compared to a shallow baseline of just 123 kNm at $H=59$ m. This phenomenon confirms that the internal structural steel rings actively absorb the primary out-of-plane bending action induced by deep or weak rock mass relaxation, protecting the adjacent precast concrete matrices from unconfined eccentric stress configurations and severe localized distortion.

The mechanical progression observed in both the longitudinal axial forces (N_1) and out-of-plane longitudinal bending moments (M_{22}) illustrates the active structural coupling and load-redistribution mechanics along the tunnel advancement axis. The steady accumulation of longitudinal thrust (N_1) away from the active TBM front reflects the three-dimensional consolidation of the ground and the transfer of machine jacking pressures through the lining tube. At the newly erected Fifth Ring Row, the structural rings operate under transient conditions with minimal longitudinal confinement, resulting in the lowest N_1 magnitudes. However, as the excavation proceeds and the annular void closes completely, the continuous rock mass mobilizes full frictional contact along the lining extrados. This interaction locks the support sequence into a monolithic tube, leading to the cumulative buildup of longitudinal axial thrust that reaches a steady-state equilibrium plateau upstream of the First Ring Row.

Simultaneously, the aggressive cyclic "sawtooth" undulation characteristic of the longitudinal bending moments (M_{22}) validates the structural shielding mechanism governed by the alternating stiffness layout. Because the structural steel rings possess a significantly higher flexural modulus than the adjacent 0.25 m concrete segments, they act as rigid circumferential restraint ribs along the longitudinal axis. When the surrounding rock mass undergoes non-uniform longitudinal deformation or differential relaxation, these high-stiffness steel rings actively intercept and isolate the resulting longitudinal bending distortions. Consequently, prominent moment peaks localize at the steel ring coordinates, while the intermediate concrete segments experience substantial flexural relief. This selective stiffness filtering alleviates severe longitudinal bending demands on the thinner concrete sections, favoring more stable joint contact conditions along the tunnel drive.

4.5.2. Torsional Effects (M_{12})

The structural evaluation of the secondary loading conditions is expanded by analyzing the maximum torsional moments (M_{12}) induced within the alternating lining layout under three-dimensional geomechanical stress configurations. Figure 4.18 and Figure 4.19 systematically illustrate the longitudinal distribution of the maximum section torsional moments across varying Geological Strength Index (GSI) parameters and operating overburden pressures (H), respectively.

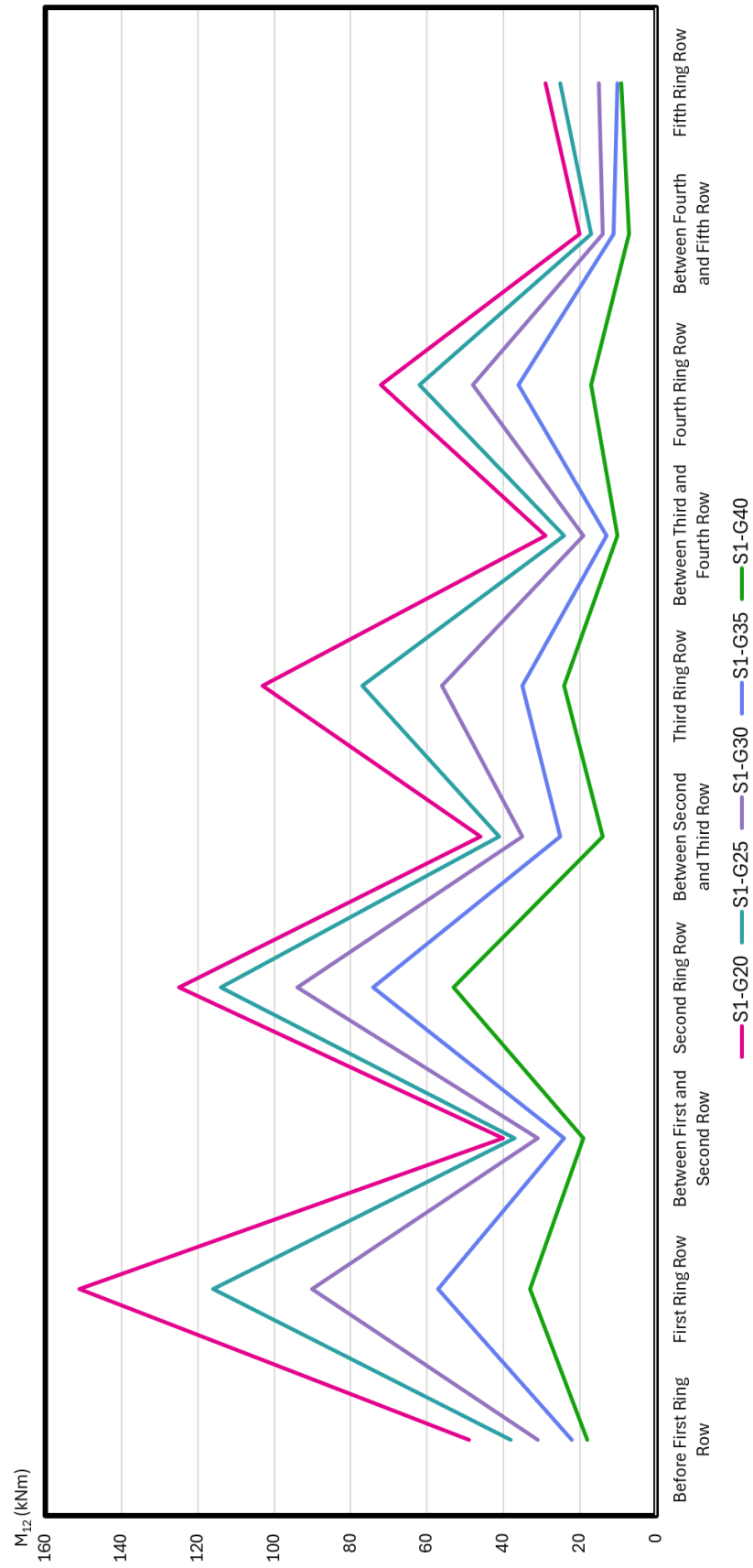


Figure 4.18 Longitudinal distribution of maximum cross-sectional torsional moments (M_{12}) across varying GSI configurations at a constant H of 99 m.

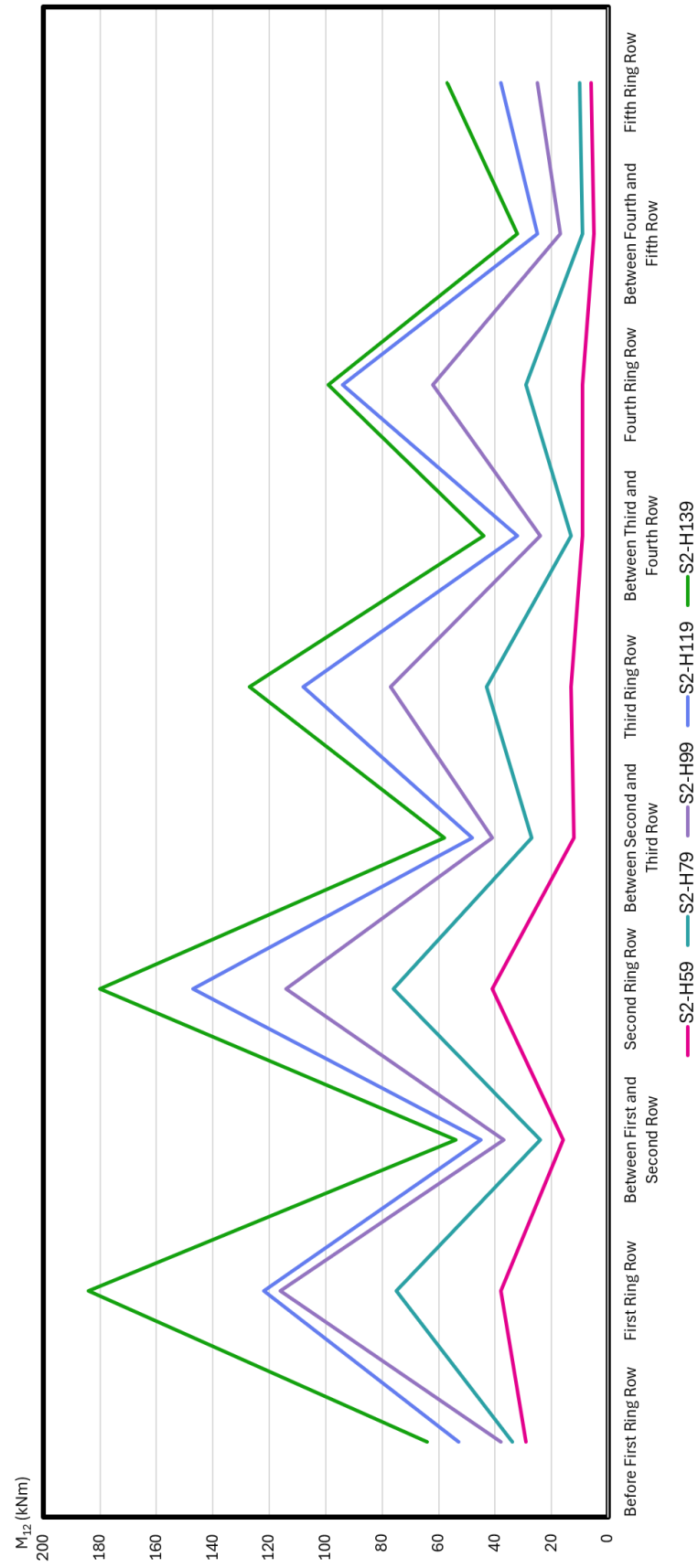


Figure 4.19 Longitudinal distribution of maximum cross-sectional torsional moments (M_{12}) across varying H configurations at a constant GSI of 25.

A macroscopic assessment of the numerical outputs establishes that the induced torsional moment fields are strongly responsive to both the geomechanical quality of the surrounding rock mass and the structural depth conditions. As the ground competence improves from a degraded state to an intermediate formation (GSI=20 to GSI=40), the maximum structural M_{12} demands drop systematically from approximately 151 kNm to 33 kNm, demonstrating a total load reduction of over 78%. Conversely, when shifting the computational modeling from a shallow configuration of $H=59$ m into a deep tunneling regime of $H=139$ m, the heightened gravitational stress fields amplify the multi-axial shear deformations, scaling the maximum structural twisting demand from a baseline of 38 kNm to a peak value of 184 kNm, which represents an accumulation increase of over 380%.

Spatial monitoring along the continuous alignment reveals that the maximum development of these torsional vectors is strictly localized within the former rings located further backward along the tunnel axis ("First Ring Row" and "Second Ring Row"). Conversely, in the sections closest to the advanced excavation zone ("Fifth Ring Row"), the induced torsional moments remain at their relative minimum boundaries. This phenomenon occurs because the freshly exposed lining elements immediately behind the shield tail clearance are partially shielded from full longitudinal shear transfer during early-stage clearance. As the excavation front advances further and the monitored sections transition toward the fully stabilized zone, the sequential lining rings achieve a fully engaged state of mechanical equilibrium with the surrounding continuous rock formation. Consequently, the induced torsional moments experience a steady, progressive accumulation across the backward simulation steps, mobilizing peak twisting actions across the established support rows.

Furthermore, the continuous longitudinal profiles of the M_{12} moments maintain a very clear, cyclic sawtooth undulation pattern driven by the structural stiffness variations across adjacent rings. The localized peaks match the exact coordinates where the high-stiffness structural steel rings are activated within the lining matrix, attracting localized twisting stresses due to their higher relative rigidity. Conversely, the local minima correspond directly to the flexible adjustments within the precast concrete segment zones. This behavioral pattern suggests that the highly rigid steel components can actively absorb and redistribute the climbing multi-axial twisting action induced by severe depth and weak rock mass relaxation. Countering these locally twisting fields, the composite

support sequence acts as a filter that substantially mitigating potential inter-segment warping tendencies and keeping structural demands within acceptable limit state boundaries under the analyzed scenarios.

The development and distribution of in-plane torsional moments (M_{12}) provide essential insights into how the composite support assembly accommodates asymmetric shear stresses and directional driving forces. The localization of peak torsional moments within the freshly installed support sequences near the active excavation zone (First and Second Ring Rows) is directly linked to the progressive engagement between the surrounding rock mass and the lining structure. During the early-age construction stage at the newly installed Fifth Ring Row, the structural components are partially shielded by the machine skin and early-stage grout consolidation, maintaining lower initial torsional demands.

As the excavation front advances further and the structural rings move into the fully engaged zone behind the shield, the closing annular gap and the continuous rock mass establish complete mechanical contact with the hardened grouted annulus. This progressive ground consolidation mobilizes long-term asymmetric shear transfers, causing the torsional moments to steadily accumulate and reach their peak profiles across the upstream support rows. Furthermore, the cyclic variation of M_{12} along the alignment demonstrates that the high-stiffness structural steel rings actively absorb the primary twisting demands induced by deep rock mass relaxation. By concentrating these multi-axial torsional stresses within the ductile steel profiles, the hybrid system effectively suppresses hazardous inter-segment warping and prevents edge chipping or localized spalling along the radial segment seams.

4.5.3. Out-of-Plane Shear Forces (Q_{13} and Q_{23})

The final part of the secondary loading state structural assessment is the study of the out-of-plane shear force fields. To do this, the longitudinal shear force component (Q_{13}) and the circumferential shear force component (Q_{23}) acting on the advanced support architecture are studied. Figure 4.20 and Figure 4.21 show the systematic longitudinal distribution of the maximum Q_{13} shear forces under different Geological Strength Index (GSI) conditions and varying operating overburden depths (H) respectively. To supplement the analysis, the progressive development of the corresponding maximum Q_{23} shear profiles is plotted in Figure 4.22 and Figure 4.23.



Figure 4.20 Longitudinal distribution of maximum cross-sectional out-of-plane shear forces (Q_{13}) across varying GSI configurations at a constant H of 99 m.

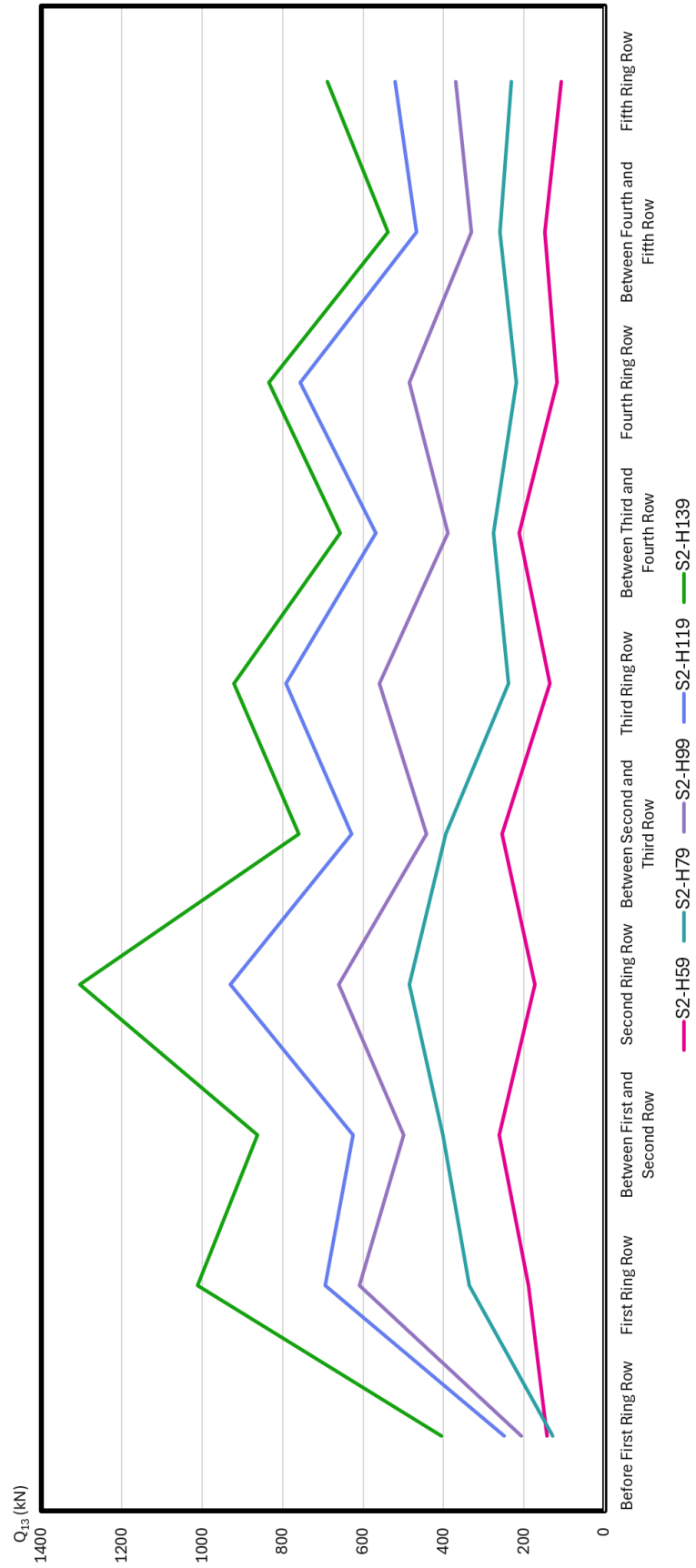


Figure 4.21 Longitudinal distribution of maximum cross-sectional out-of-plane shear forces (Q_{13}) across varying H configurations at a constant GSI of 25.

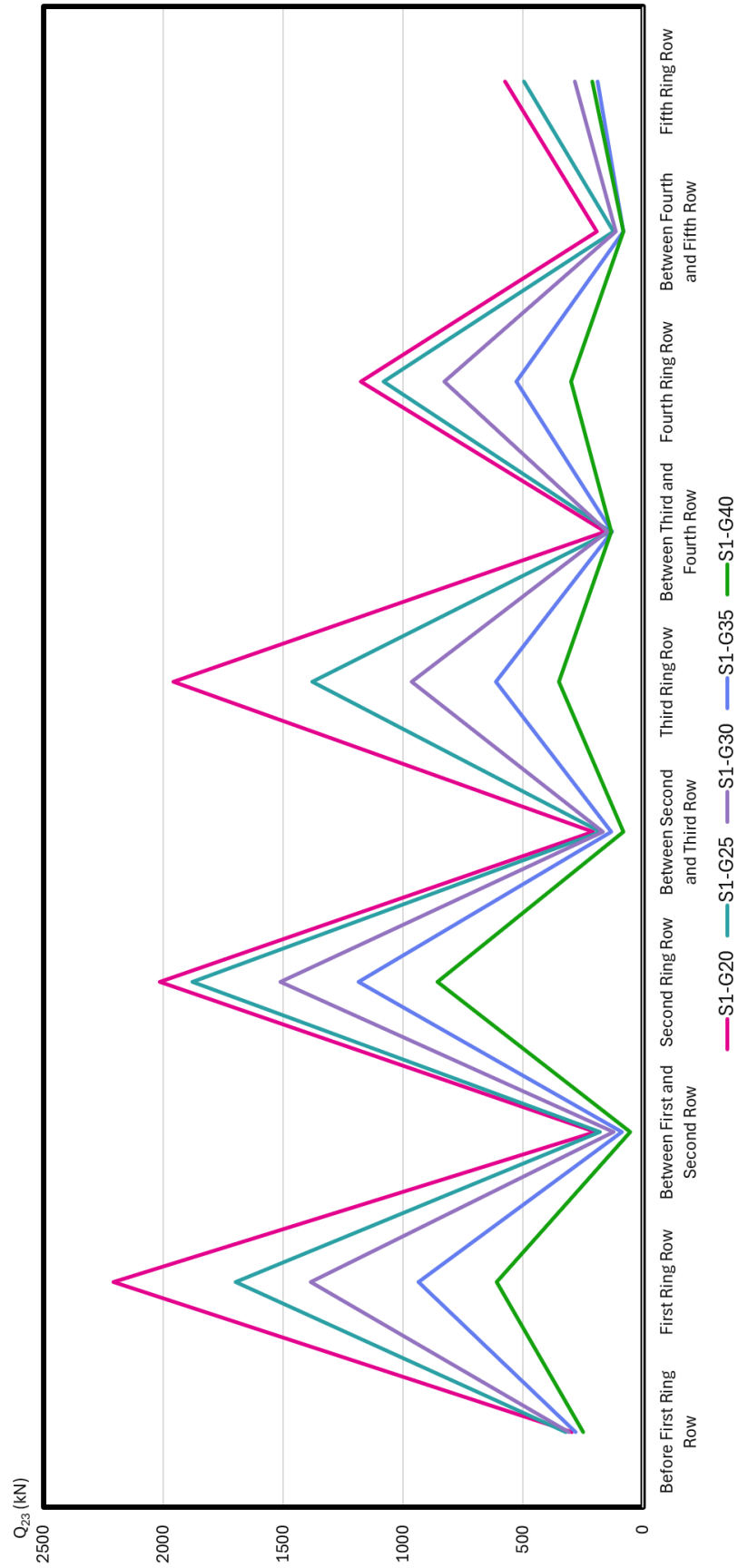


Figure 4.22 Longitudinal distribution of maximum cross-sectional out-of-plane shear forces (Q_{23}) across varying GSI configurations at a constant H of 99 m.

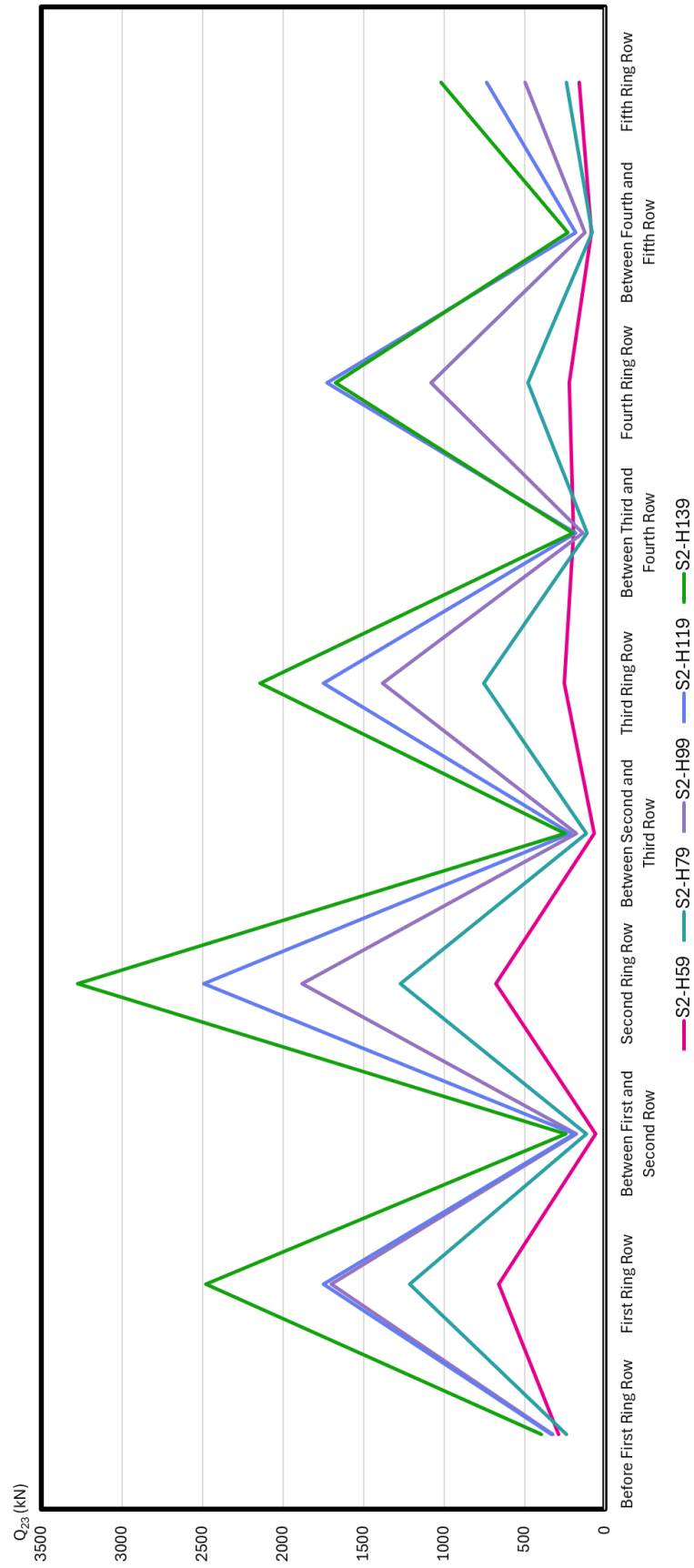


Figure 4.23 Longitudinal distribution of maximum cross-sectional out-of-plane shear forces (Q_{23}) across varying H configurations at a constant GSI of 25.

A macroscopic assessment of the computational outputs demonstrates that both the Q_{13} and Q_{23} structural force fields exhibit extreme sensitivity to the geomechanical competence of the rock mass and the operating depth environment. Based on the numerical datasets, as the ground quality transitions from highly degraded to intermediate regimes (GSI=20 to GSI=40), the peak longitudinal out-of-plane shear forces (Q_{13}) captured along the First Ring Row drop systematically from 789.00 kN to 236.00 kN. Simultaneously, the corresponding circumferential out-of-plane shear forces (Q_{23}) within the same section experience a severe reduction from 2209.00 kN under poor ground conditions (GSI=20) down to 610.00 kN under enhanced ground competence configurations (GSI=40).

Conversely, scaling the depth configuration from $H=59$ m into a deep tunneling regime of $H=139$ m expands the total gravitational stress fields transferred to the composite assembly. This mechanical transition escalates the maximum structural Q_{13} forces within the Second Ring Row from a shallow baseline of 173.00 kN to a substantial peak of 1305.00 kN. Following an identical behavioral trend, the maximum structural Q_{23} demand along the Second Ring Row scales from 675.00 kN under the shallow regime up to a substantial peak value of 3275.00 kN under the maximum geostatic depth of $H=139$ m.

Spatial tracking along the continuous monitoring trajectory reveals that both out-of-plane components display a prominent, cyclic sawtooth fluctuation pattern across all simulation stages. This recurring mechanical trend is directly governed by the intense cross-sectional stiffness contrast inherent to the 3+1 alternating structural layout. Rather than showing a monotonic dissipation toward the tunnel face, the structural forces experience heavy periodic accumulations at specific ring rows. The maximum localized peaks are systematically concentrated within the First Ring Row and Second Ring Row configurations, where the high-stiffness structural steel rings are explicitly activated within the lining matrix. The increased relative stiffness of these steel elements, naturally allows them to intercept, attract and safely withstand the multi-axial shear deformations transmitted from the surrounding continuous rock mass. The behavior of this load sharing clearly shows that the addition of the rigid structural steel rings substantially relieves out-of-plane shear concentrations in the adjacent concrete panels, maintaining localized eccentric demands at manageable levels and limiting the potential for significant joint dislocations under the analyzed loading states.

The out-of-plane shear force fields (Q_{13} and Q_{23}) characterize the transverse structural shear transfer and punching resistance mobilized across the alternating support layout under severe three-dimensional geostatic stresses. The sharp escalation in both shear components under deep overburden regimes ($H = 139$ m) and degraded rock conditions ($GSI = 20$) is driven by the severe disparity between vertical lithostatic overburden and horizontal lateral confinement. In low-GSI rock masses, the lack of joint interlocking triggers extensive plastic yielding around the cavity, producing concentrated radial shear stresses that act perpendicular to the lining shell.

The pronounced cyclic sawtooth undulation observed across both Q_{13} and Q_{23} profiles highlights the critical role of the structural steel rings in bridging localized shear concentrations. Because of the sudden stiffness transition between the 0.25 m concrete panels and the high-modulus steel elements, the structural steel rings act as stiff transverse load-bearing diaphragms that actively intercept and absorb the primary out-of-plane shear demands. This load-transfer capability reduces the potential for localized punching shear distress and differential radial stepping between adjacent rings, contributing to a more coherent structural response within acceptable limit state boundaries even under the most demanding geostatic pressure fields.

4.6. Summary of All Scenarios Results

This section provides a structured comparative assessment of the hybrid 3+1 lining architecture by synthesizing the numerical datasets obtained across the entire parametric modeling matrix. Rather than evaluating isolated cross-sectional metrics, the progressive evolution of internal lining forces and deformation profiles is cross-examined to systematically establish the direct correlations between external geomechanical variables and the composite structural response of the alternating segments. The comprehensive analysis confirms that the mechanical behavior of the hybrid system is governed by a coupled interaction between the competence of the surrounding rock mass and the operating overburden depth, rather than by either parameter acting as an independent variable. While the Geological Strength Index (GSI) acts as the primary governing factor dictating the rock mass self-supporting capacity and the prevailing load-transfer mechanism, the operating overburden depth (H) serves as the primary scaling amplifier for the absolute geostatic stress field. Evaluating these interrelated parameters within a

unified framework provides a rigorous foundation for defining the operational boundaries and load-shielding effectiveness of the alternating stiffness configuration.

4.6.1. Effect of GSI

The geomechanical competence of the surrounding rock mass, expressed by Geological Strength Index (GSI), is the main controlling factor of the overall load-transfer mechanism. In heavily degraded ground regimes (GSI = 20), the absence of stable ground arching actions causes the lining assembly to directly sustain severe geostatic load accumulations, which increases the structural utilization indexes to be close to the design limit states. Conversely, as the ground quality improves toward intermediate properties (GSI = 40), the formation of an effective rock mass arch shields the structural profile. This improvement triggers a sharp reduction in structural demands, attenuating the sensitivity of the internal forces and confirming that higher ground competence significantly reduces the structural dependency on heavy structural steel support variations.

The physical mechanisms underlying this pronounced sensitivity to GSI variations are directly governed by the development and stability of the natural ground arch around the excavated cavity. In fair-to-intermediate rock masses characterized by GSI = 40, the presence of tight discontinuity interlocking and high intact shear resistance enables the formation of an effective self-supporting rock ring directly ahead of and around the advancing shield. This geological arch intercept and transfers the primary geostatic pressures away from the cavity perimeter, allowing the installed 0.25 m lining to operate under minimal structural demands. Under these favorable ground conditions, the need for auxiliary internal stiffening is minimized because the surrounding rock mass acts as the primary load-bearing element of the tunnel stabilization scheme. Conversely, in poor and disintegrated rock masses (GSI = 20), the absence of structural interlocking triggers extensive plastic yielding and unconfined rock relaxation immediately upon shield tail clearance. The natural ground arch fails to form, transforming the structural problem from one of displacement containment into direct gravity load resistance where the composite lining assembly must independently support the full weight of the overlying ground column. This phenomenon explains the steep non-linear surge in axial forces, bending moments, and shear actions observed across the low-GSI models, indicating that the

structural shielding provided by the internal steel rings is most intensely mobilized and plays a highly critical role in weak rock formations within the studied parameter range.

4.6.2. Influence of Overburden Depth

The operating overburden depth (H) directly dictates the magnitude of the gravitational and lateral stress fields transferred to the composite support sequence. Scaling the depth configuration from the baseline shallow regime of $H = 59$ m to the peak design boundary of $H = 139$ m induces a proportional expansion in all internal structural force metrics. Under these deep tunneling conditions, the severe horizontal stress concentrations shift the primary structural normal stress and bending demands heavily toward the lateral sidewalls of the lining assembly. The numerical outputs demonstrate that while shallow configurations maintain conservative structural fields across all segments, the maximum depth transitions require the full activation of the alternating structural steel matrices to balance the severe pressure expansions without localized concrete crushing.

The structural response to depth scaling demonstrates that increasing the operating overburden depth (H) from 59 m to 139 m does not merely amplify the absolute magnitude of internal forces, but substantially increases the cross-sectional stiffness contrast and load-sharing dynamics between the lining components. Under shallow tunneling conditions ($H = 59$ m), the moderate in-situ stress state produces relatively low and uniform force distributions across both the concrete segments and the steel rings, resulting in modest peak-to-valley differentials along the alignment. However, as the depth escalates toward deep tunneling regimes ($H = 139$ m), the severe gravitational and lateral geostatic confinement mobilizes the full composite action of the support sequence. Due to their significantly higher elastic modulus (210 GPa) and shear rigidity, the structural steel rings attract a disproportionately higher share of the depth-induced stresses, as evidenced by the dramatic expansion in the bending moment differential between the steel rings and adjacent concrete panels. Furthermore, the high lateral earth pressures developed at great depths shift the primary hoop thrust (N_2) and flexural moments (M_{11}) heavily toward the lateral sidewalls (springlines), while in-plane shear forces (Q_{12}) concentrate along the vertical crown-invert axis. This clear directional separation of load-bearing actions ensures that the adjacent 0.25 m concrete segments remain confined within safe compressive thresholds without experiencing excessive

eccentric flexure or brittle crushing, verifying the viability of the hybrid system under deep overburden conditions.

It should be noted that the linear escalation of lining forces with depth observed here is closely tied to the weak-to-intermediate ground conditions considered ($GSI \leq 40$). In highly competent rock masses characterized by higher GSI values, increased overburden depth can actually act favorably by mobilizing confining stresses and forming a robust self-supporting ground arch. Because this parametric study focused on ground regimes with limited natural arching capacity, such favorable deep-arching mechanisms fall outside the measured scope of this work and could yield different lining demand trends in very competent rock.

4.7. Best Structural Performance Conditions

Following the multi-parametric cross-examination, the best structural performance conditions of the proposed hybrid 3+1 lining architecture can be distinctly identified. The structural layout is optimally exploited when deep modeling environments are combined with weak to intermediate ground regimes, where the contrast in stiffness of the structural steel rings and standard concrete panels is fully exploited.

The main operational advantage of this alternating design is its special load-redistribution capacity. The vertical axis high-stiffness steel elements efficiently isolate and absorb the intense in-plane shear forces (Q_{12}) that accumulate at the tunnel crown and invert coordinates, under peak overburden pressures. At the same time the lateral concrete sections take care of the large build-ups of circumferential thrust resulting from the horizontal pressure fields. This coordinated structural response prevents localized plastic distress, limits detrimental lining ovalization and ensures a balanced stress state. Therefore, the hybrid 3+1 design is optimized to a great degree for the challenging geomechanical conditions, with full structural resilience well below conservative ultimate limit state thresholds.

A comprehensive cross-sectional investigation conducted under the peak design boundary conditions further clarifies the exceptional safety margins and directional efficiency of this hybrid architecture. Despite the severe magnitude of the multi-axial geostatic pressure fields encountered under the worst-case scenario combining maximum depth ($H = 139$ m) with degraded ground conditions ($GSI = 20$), the normalized internal

force utilization metrics across both the standard concrete panels and the alternating steel configurations were found to remain within the conservative 0.60 performance threshold under the investigated conditions. This robust capacity margin is directly enabled by the specialized, orthogonal load-sharing regime developed within the composite ring: the high-stiffness structural steel elements at the crown and invert intercept the symmetric in-plane shear peaks (Q_{12}), while the lateral concrete panels mobilize the heavy horizontal confinement to resist circumferential hoop thrust (N_2) and flexural moments (M_{11}) under predominantly uniform compression.

The critical engineering significance of maintaining the force utilization ratios below 0.60 lies in the prevention of localized concrete micro-cracking and long-term plastic fatigue along the longitudinal joints. In deep mechanized tunneling, high compressive eccentricities often lead to corner spalling and longitudinal splitting at segment interfaces. By utilizing the internal structural steel rings as rigid circumferential restraint ribs, the composite system successfully absorbs the peak flexural and shear distortions, keeping the concrete matrix in a favorable stress state. This coordinated structural response limits localized plastic yielding, restrains excessive lining ovalization, and indicates that the 0.25 m thin-walled hybrid configuration can maintain adequate operational stability and structural redundancy within standard limit state design boundaries for the simulated cases.

4.8. Material Optimization and Sustainability Implications

The demonstrated capability of the hybrid 3+1 architecture to strategically redistribute internal stress concentrations and isolate localized peaks carries profound implications for material optimization and sustainable tunnel engineering. Conventional underground construction practices typically address extreme geomechanical relaxation fields and high geostatic loading environments by enforcing a uniform increase in lining thickness or specifying ultra-high-performance concrete grades along the entire continuous drive. While this traditional, mass-centric design approach ensures structural capacity, it results in massive material waste, inflated procurement costs, and an excessively heavy environmental carbon footprint, which directly contradicts modern sustainable engineering directives.

The multi-parametric numerical data compiled across this study demonstrate that because the absolute peak structural demands are successfully isolated and concentrated within specific alternating coordinates, the structural capacity of the standard precast concrete panels does not need to be uniformly over-designed. By maintaining a standardized, thin cross-section of 0.25 m for the concrete segments and deploying high-stiffness structural steel rings in an alternating sequence, the structural performance achieved satisfies all design limit states without requiring the typical 0.40 m thickness common in deep alignments. This strategic allocation of structural stiffness yields an immediate 37.5% reduction in the total volume of concrete required per linear meter of TBM excavation. Consequently, the overall excavation diameter of the tunnel bore is systematically decreased, which directly minimizes the total volume of excavated muck, lowers spoil haulage demands, and reduces environmental disposal footprints.

From a lifecycle and sustainability perspective, this drastic reduction in concrete consumption translates directly into substantial savings in embodied carbon emissions, as cement manufacturing remains one of the primary industrial contributors to global greenhouse gas production. Furthermore, the reduction in the dead weight of the individual precast segments mitigates critical operational and ergonomic constraints during the construction phase. Lighter segments decrease mechanical stress and hydraulic demand on TBM segment erectors, simplify shaft lifting and underground transport logistics, and reduce the likelihood of accidental corner chipping or spalling during handling and ring assembly. Therefore, the hybrid 3+1 configuration demonstrates the technical feasibility of decoupling structural resilience from material mass, offering a promising and resource-efficient framework for low-carbon mechanized tunneling in challenging geological environments.

5. CONCLUSION AND RECOMMENDATIONS

This chapter synthesizes the primary findings, engineering evaluations, and design implications established through the three-dimensional finite element parametric simulations of the hybrid 3+1 segmental tunnel lining system. By systematically investigating the coupled interactions between rock mass competence and operating overburden depths, the operational boundaries, load-redistribution mechanics, and capacity utilization limits of the thin-walled, steel-reinforced configuration are characterized. The following sections provide detailed discussions on the key findings, formulate concrete answers to the foundational research questions, document the specific scientific and practical contributions of this work, and define the recognized limitations alongside recommendations for future research.

5.1. Key findings

The numerical investigation demonstrates that the integration of high-stiffness structural steel rings into a 0.25 m thick concrete segmental lining establishes an active, longitudinal load-redistribution mechanism along the tunnel drive. Possessing an elastic modulus of 210 GPa, the structural steel rings attract disproportionately higher normal forces and flexural demands compared to the adjacent concrete sections. This difference in stiffness leads to a characteristic cyclic sawtooth distribution of all primary structural force measures, with well-pronounced demand peaks localizing at the steel ring positions and significant stress relief occurring within the intermediate concrete segments. Therefore, the high-stiffness rings are effective in shielding the thin concrete sections from excessive stress concentrations and unconfined eccentric loading, supporting the mechanical feasibility of the hybrid support configuration under the evaluated conditions.

The internal lining forces are controlled mainly by the geomechanical competence of the rock mass, which is characterized by the Geological Strength Index. Under a constant overburden depth of 99 m, degrading the rock mass quality from $GSI = 40$ down to $GSI = 20$ causes the maximum cross-sectional hoop thrust (N_2) to escalate from 11920 kN to 27500 kN at the Second Ring Row, while the maximum in-plane bending moment (M_{11}) increases from 70 kNm to 205 kNm. In competent formations, the development of an effective natural ground arch bears a substantial portion of the geostatic pressure, keeping lining demands well-controlled. In contrast, the lack of stable interlocking in low GSI

regimes leads to unconfined ground relaxation immediately behind the shield, transferring a substantially higher portion of the geostatic overburden directly to the composite lining assembly.

The operating overburden depth serves as the direct scaling driver for absolute structural demand fields. Varying the depth from 59 m to 139 m under uniform ground conditions ($GSI = 25$) increases the peak hoop axial force at the Second Ring Row from 9281 kN to 39770 kN and the peak in-plane bending moment from 55 kNm to 309 kNm. The interesting point is that the amplitude of the cyclic sawtooth undulation increases significantly with depth, the difference in bending moment between the steel ring and the concrete segments in the vicinity increases from 13 kNm at $H = 59$ m to 94 kNm at $H = 139$ m. This shows that the load-sharing and shielding efficiency of the steel rings is more intensely mobilized under high geostatic confinement. Furthermore, while in-plane shear forces (Q_{12}) reach their maximum values of up to 1527 kN symmetrically along the vertical axis at the tunnel crown and invert, hoop compression (N_2) and flexural moments (M_{11}) concentrate along the lateral sidewalls (springlines), confirming a specialized, directional load-bearing regime within the composite ring.

5.2. Answers to Research Questions

Numerical results indicate that structural stability can be largely preserved for thin-walled 0.25 m segments in deep tunneling regimes, at overburden depths up to 139 m, when supported by periodic internal steel rings. Despite the severe magnitude of geostatic pressures encountered in deep Hoek-Brown rock formations, the structural force utilization ratios across both the steel rings and the concrete panels were observed to remain within the conservative 0.60 design limit state threshold under the investigated scenarios. The uniform compressive resistance develops along the sidewall segments due to the continuous lateral confinement, preventing brittle failure and limiting cross-sectional ovalization.

The parametric data sets reveal that rock degradation leads to an exponential increase in the normal forces and flexural actions as the hybrid system's reaction to the degrading rock mass quality. The reduction of GSI from 40 to 20 at a constant depth of 99 m causes an increase of 130% in maximum axial forces and 192% in in-plane bending moments.

This structural intensification is attributed to the degradation of joint conditions and interlocking of intact blocks which eliminates the self-supporting ground arch and converts the support problem from displacement containment to direct gravity load resistance.

The load-sharing behavior between concrete and steel evaluated through depth scaling indicates that increasing overburden depth mobilizes the composite action of the system more effectively. The rate of force absorption of steel rings under increasing geostatic pressure is better than that of concrete segments, because the elastic modulus and shear stiffness of steel rings are better. The internal steel bracing provides a stiff structural frame which takes up the main transverse and longitudinal shear distortions, so that the adjacent concrete sections can work under a largely uniform compressive state with very little eccentricity of bending.

5.3. Contribution to Science

The main scientific contribution of this thesis is the development and numerical validation of a performance-oriented design framework for segmental tunnel linings. This research challenges the conventional mass-centric paradigm of mechanized tunneling by demonstrating that high structural resilience can be achieved through targeted stiffness adjustments rather than by increasing the concrete mass uniformly. The results demonstrate the feasibility of decoupling structural capacity from uniform lining thickness. This suggests that thin-walled segments, when combined with periodic auxiliary steel elements, present a viable structural alternative for deep and adverse ground conditions within the analyzed operational envelope.

Furthermore, the present study provides a detailed and uniform numerical database describing the 3D mechanical interactions of a 3+1 hybrid lining architecture. The work systematically quantifies the longitudinal and circumferential redistribution profiles of primary forces (N_2 , M_{11} , Q_{12}) as well as secondary multi-axial actions (N_1 , M_{22} , M_{12} , Q_{13} , Q_{23}) across a broad parametric matrix. By correlating these force profiles with specific combinations of GSI and overburden depth, the study establishes validated operational envelopes that fill an existing empirical and numerical gap in the tunneling literature regarding the structural limits of thin segments under severe geostatic stress fields.

5.4. Practical Recommendations for Tunnel Designers

From a practical design perspective, the application of the hybrid 3+1 lining configuration should be targeted primarily at deep tunneling projects ($H > 100$ m) driven through weak to intermediate rock masses ($GSI < 30$). In highly competent rock formations characterized by GSI values of 40 or greater, the natural rock arching mechanism carries the vast majority of geostatic pressures; consequently, standard thin-walled concrete linings without auxiliary steel rings appear structurally adequate, and deploying steel rings in such competent ground offers limited technical necessity. A rigorous geomechanical screening should therefore precede support selection to deploy the hybrid architecture exclusively in geotechnical zones where severe squeezing or heavy overburden demands justify the intervention.

Structural detailing must account for the directional concentration of internal forces identified along the lining perimeter. The joints between the radial segments in the zones where in-plane shear forces (Q_{12}) are always maximum along the vertical axis at the crown and invert of the tunnel have to be reinforced with high-capacity shear dowels, alignment pins or heavy interlocking keyways to avoid localized shear slippage and concrete chipping. Simultaneously, while the peak circumferential compression (N_2) and bending moments (M_{11}) are concentrated along the lateral sidewalls, it is recommended to incorporate compressible packing materials or high-durability elastomeric bearing pads along the springline radial joints to promote uniform contact stress distribution and prevent premature edge spalling. Furthermore, the longitudinal tie rods flanking the steel-to-concrete interfaces should be specified with high tensile capacity and calibrated pre-stressing to withstand the cyclic longitudinal bending moments (M_{22}) and preserve joint watertightness.

To translate the numerical findings into comprehensive detailing specifications, specific attention must be paid to the following engineering design measures:

- **Zonal Shear Detailing along the Vertical Axis:** Given that peak in-plane shear forces (Q_{12}) systematically concentrate at the tunnel crown and invert under deep conditions, circumferential joints at these antipodal locations must be detailed with specialized shear-transfer keys or high-

grade steel guide rods. These elements ensure continuous transverse shear transmission across ring boundaries, preventing localized radial displacement steps or damage to tail skin sealing brushes during shield advance.

- Sidewall Joint Packing and Eccentricity Management: Because peak hoop thrust (N_2) and flexural moments (M_{11}) consistently align along the springlines, radial joints flanking the sidewall segments require high-durability elastomeric packing pads. These inserts distribute the intense compressive resultant evenly across the contact area, accommodating minor erection misalignments and eliminating the risk of concrete bursting or corner spalling under deep geostatic confinement.
- Longitudinal Connector Optimization: The severe cyclic sawtooth fluctuation documented in out-of-plane longitudinal bending moments (M_{22}) indicates high structural boundary interactions at the interfaces between concrete segments and structural steel rings. Connector bolts and longitudinal tie rods traversing these interfaces should be engineered with enhanced shear and tensile capacities, supplemented by calibrated pre-tensioning to maintain tight gasket compression and guarantee long-term watertightness against high external water tables.
- Alignment Geomechanical Screening: Deploying the hybrid 3+1 lining should be governed by a rigorous geological classification protocol during excavation planning. Because competent rock formations ($GSI \geq 40$) mobilize sufficient natural ground arching, standard precast concrete rings remain fully adequate without steel reinforcement. Restricting the installation of the steel-reinforced hybrid configuration primarily to critical geotechnical reaches ($GSI < 30$ and $H > 100$ m) balances economic efficiency and operational advance rates while maintaining acceptable structural safety margins.

5.5. Limitations of the Study

Several computational and physical simplifications inherent to the numerical modeling methodology must be acknowledged. The first step was to model the precast concrete

segments and the structural steel rings as isotropic linear elastic plates elements. While providing a good framework for the evaluation of structural force redistribution and initial capacity utilization, this approach does not explicitly model non-linear material degradation, concrete tensile cracking, post-yield plastic deformation of steel, or crushing behavior under severe localized stress concentrations.

Second, the continuous shell formulations were applied to the simulation of structural interactions of adjacent rings and single segments without explicit representation of discrete, frictional joint interfaces. Thus, the effects of gasket compression, partial joint gapping, bolt shear deformation and variations in the rotational joint stiffness were not fully resolved. Moreover, the numerical simulations were strictly limited to a static continuum formulation, thereby excluding dynamic shockwaves, earthquake-induced seismic ground shaking, and transient construction impacts. Additionally, the rock mass deformation modulus was kept constant across GSI variations to limit the number of input variables. Finally, the geotechnical domain was modeled assuming homogeneous and isotropic rock mass conditions governed by the Hoek-Brown criterion, without explicitly incorporating discrete joint sets, fault zones, or time-dependent rheological effects such as rock squeezing, concrete creep, and shrinkage. Furthermore, the parametric study was restricted to poor-to-moderate rock masses ($GSI \leq 40$); consequently, potentially favorable ground arching mechanisms that could develop under deep cover in higher GSI conditions were outside the measured scope and could result in different structural trends.

5.6. Future Research Directions

Physical laboratory testing is needed to experimentally validate the numerical results and the load sharing mechanisms observed in this study. Full-scale or scaled beam-spring and ring-assembly tests subject to non-uniform radial loading frames would provide empirical data on contact mechanics, localized stress concentrations, and failure modes at the steel-concrete interface. Such experimental programs are critical to calibrate numerical friction coefficients and verify joint detailing under controlled physical conditions.

Future computational frameworks should include advanced time dependent constitutive formulations to describe the rheological behavior of both the support elements and the surrounding rock mass. Viscoelastic modeling frameworks should be used to study the

long-term redistribution of internal forces induced by concrete creep, autogenous shrinkage and progressive rock squeezing during the design life of tunnel. The evaluation of these effects is important to verify the steady long-term deformation fields if the steel rings have a stable load-shielding efficiency.

The dynamic and seismic loading conditions should be considered to study the performance of the hybrid 3+1 lining configuration. The transient response of the alternating stiffness system should be studied by the dynamic time history analysis in three dimensions by using recorded accelerograms of earthquakes. Special attention should be paid to the evaluation of the shear transfer capacity between the steel rings and concrete segments under the ground ovalization and the axial racking deformations caused by the seismic action.

Future numerical studies should use advanced non-linear joint models, which explicitly simulate the physical interfaces of the adjacent concrete segments and steel rings. The opening of the joints, contact stress concentrations, and the risk of leakage under high eccentric loading can be followed in detail bridging the gap of continuum shell analysis and the discrete structural reality using surface-to-surface contact pairs with Coulomb friction, non-linear rotational springs and elastomeric gasket models.

complete life-cycle assessment (LCA) and cost-benefit analysis should be conducted to determine the full economic and environmental viability of the hybrid lining system. This research should quantify the trade-offs between the 37.5% reduction in concrete volume, reduced excavation diameter, and lower muck disposal requirements and the costs of procurement, fabrication, corrosion protection, and underground erection of the structural steel rings. Such an evaluation would provide contractors and project owners with clear metrics on the net carbon savings and capital expenditure impacts of the proposed optimization framework.

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APPENDICES

APPENDIX 1 – Publication Related to the Thesis

APPENDIX 2 – Thesis Study Originality Report