

Installation-Delay Effects on Passive-Bolt Anchorage in Squeezing Railway Tunnels: An Analytical Framework for Support Transition and Cost Optimization

Huy Nghia Nguyen

École Spéciale des Travaux Publics

Truyen Tran

tranthetruyen@utc.edu.vn

University of Transport and Communications

Xuan Tung Nguyen

Campus Hochiminh city, University of Transport and Communications,

Minh Ngoc Vu

Duy Tan University

Article

Keywords: Squeezing ground, railway tunnel, convergence-confinement method, passive rock bolt, support-installation delay, techno-economic optimization

Posted Date: September 24th, 2026

DOI: <https://doi.org/10.21203/rs.3.rs-10922234/v1>

License: © ⓘ This work is licensed under a Creative Commons Attribution 4.0 International License.

[Read Full License](#)

Additional Declarations: No competing interests reported.

Abstract

Support installation in squeezing railway tunnels is rarely instantaneous, yet conventional convergence-confinement analyses often treat support components as if they mobilize simultaneously. Here we couple the convergence-confinement method with component-specific anchorage criteria and a normalized techno-economic model to examine how installation delay changes the relative performance of passive rock bolts and shotcrete. The framework tracks stress release, pre-support plastic-zone development and effective bolt embedment across variable rock-mass quality and installation conditions. As rock quality deteriorates, the plastic annulus can exceed the 2.75 m effective embedment of 3.95 m passive bolts before a 150 mm shotcrete lining reaches its nominal capacity. The model places the intervention band near RMR 48.2–50.1. For RMR = 35, continuous optimization gives a 9.61 m cable-anchor length; a field-standard 10 m tendon gives an estimated as-built cost of 3105.4 CU/m. Back-analysis against published Driskos Twin Tunnel monitoring reproduces the overall longitudinal deformation trend. The framework is intended for preliminary screening of support transitions rather than final design, and explicitly retains the limitations of a one-dimensional, time-independent CCM formulation.

1. INTRODUCTION

Excavation of large-span tunnels through tectonically disturbed rock requires careful control of stress redistribution, particularly at substantial overburden. The issue is relevant to high-speed railway (HSR) corridors such as the proposed North-South HSR in Vietnam, for which support performance and construction cost must be assessed at the feasibility and design stages [1, 2]. Although the New Austrian Tunneling Method (NATM) seeks to mobilize the inherent strength of the rock mass through staged excavation and timely support, the response at an advancing face remains difficult to predict. This uncertainty is amplified during early design because geomechanical indices such as the Geological Strength Index (GSI) and Rock Mass Rating (RMR) are spatially variable and are often represented by deterministic values [3, 4].

The Convergence-Confinement Method (CCM) provides a widely used analytical representation of ground-support interaction [5]. When combined with the generalized Hoek-Brown criterion, the method can define the elastoplastic response of fractured rock masses [6, 7]. Conventional applications, however, commonly idealize the support system as if its components mobilized resistance synchronously. Field observations and recent numerical studies indicate that failure in deeply buried or squeezing ground can be strongly component-dependent [8–11]. In particular, expansion of the plastic annulus may eliminate the stable embedment required for passive rock bolts to develop pull-out resistance before early-age shotcrete reaches its structural limit [12, 13].

Recent optimization studies have used finite-element and artificial-intelligence-assisted models to improve tunnel-support design [14–16]. Many such studies nevertheless adopt idealized construction sequences and do not explicitly link support-installation timing to deformation and direct cost. In

practice, delays during muck removal, rock-bolt installation, shotcreting, or invert closure increase stress release before confinement becomes effective. This gap between idealized structural analysis and field execution can lead to additional expenditure without controlling deep-seated deformation [11, 17]. Once the plastic zone approaches the available bolt embedment, deeper prestressed cable anchorage may therefore be more effective than densifying a short passive-bolt pattern [18, 19].

This study develops an integrated mechanical and economic framework for preliminary support selection in large-span HSR tunnels. This analysis evaluates only the stress-driven elasto-plastic yielding of the support structures. Bypassing complex time-dependent parameters and pore water pressure fluctuations yields a simplified but robust computational baseline for cost estimation. A bounded stress-release factor is introduced into the CCM to represent variability in support-installation timing and to distinguish the deformation-controlled response of passive rock bolts from the stress-controlled response of shotcrete. A normalized cost-unit (CU) model, calibrated using Vietnamese construction norms and market databases, is then used to compare alternative support strategies while reducing the influence of currency and inflation [2, 20–22]. The analysis finally combines geological condition and execution delay in a bivariate risk-cost matrix to identify the transition from conventional passive bolting to deep active anchorage.

2. METHODS AND ANALYTICAL FRAMEWORK

Tunnel stability and support demand are evaluated using the Convergence-Confinement Method (CCM) [5]. The method provides an analytical relationship between ground deformation and the pressure mobilized by the support system and is consistent with the staged-support concept of NATM. The formulation adopted here combines rock-mass characterization, equivalent strength parameters, elastoplastic ground response, support stiffness, and a normalized cost model.

2.1. Material models and support-design criteria

Rock-mass quality is characterized using the RMR system [1, 4]. The investigated alignment includes weathered and discontinuous rock ranging from the baseline fair-rock condition to poorer Class IV-V zones. Intact-rock properties, including uniaxial compressive strength (σ_{ci}) and the Hoek-Brown material constant (m_i), are obtained from laboratory testing in accordance with ISRM procedures.

A key input is the rock-mass deformation modulus (EM). Correlations developed for competent rock can overestimate stiffness when applied to fractured or poor-quality rock [23]. Because the critical scenarios in this study include RMR values not exceeding 56, the exponential relation proposed by Serafim and Pereira [24] is adopted. This relation represents the rapid stiffness reduction associated with deteriorating rock quality and is consistent with the nonlinear convergence observed in squeezing tunnels [7, 13, 25]:

$$E_M = 10^{\frac{RMR-10}{40}} \quad (GPa)$$

where E_M is the rock-mass deformation modulus and RMR is the Rock Mass Rating.

The rock mass is represented as an equivalent continuum governed by the generalized Hoek-Brown criterion. The parameters m , s , and a are derived from the Geological Strength Index (GSI), which is assumed to be equivalent to RMR for $RMR > 23$; this relationship is applied only within the stated range and should be interpreted as a preliminary-design approximation [1, 26]. The adopted input parameters are summarized in Table 1.

Table 1
Baseline geometry, geotechnical and support parameters.

Category	Parameter	Value	Unit	Note
Geometry	Tunnel span, B	13	m	-
Geometry	Tunnel height, Ht	10	m	-
Geometry	Excavation area	111.87	m ²	-
Geometry	Equivalent radius, R0	5.97	m	-
Geometry	Overburden depth, H	63.77	m	Reference section
Geotechnical	Unit weight, γ	25.7	kN/m ³	-
Geotechnical	Internal friction angle, φ	33	°	-
Geotechnical	Cohesion, c	0.25	MPa	-
Geotechnical	Poisson ratio, ν	0.15	-	-
Geotechnical	Uniaxial compressive strength	> 25	MPa	-
Geotechnical	Rock Mass Rating	59	-	Baseline
Geotechnical	Intact-rock modulus	50	GPa	-
Shotcrete (B25/M30)	Layer thickness	0.15	m	-
Shotcrete (B25/M30)	Compressive strength	30	MPa	-
Shotcrete (B25/M30)	Elastic modulus	29.44	GPa	-
Shotcrete (B25/M30)	Poisson ratio	0.20	-	-
Rock bolts	Bolt type	Swellex	-	-
Rock bolts	Diameter	25	mm	-
Rock bolts	Length	3.95	m	-
Rock bolts	Spacing	1.44	m	-
Rock bolts	Maximum support pressure	0.342	MPa	-

For better-quality rock ($RMR > 56$), the rock-mass modulus is estimated from the alternative relation reported by Bieniawski [1] and Lowson and Bieniawski [25]:

$$E_M = 14 + (E_i - 14) \times \left[1 - \left(\frac{100 - RMR}{44} \right)^{\frac{RMR}{70}} \right] (GPa)$$

where E_i is the deformation modulus of the intact rock (GPa).

The equivalent cohesion c and friction angle φ are obtained from the adopted strength data and the RMR-dependent reduction procedure used in the design model.

The minimum primary rock-bolt length is estimated using the empirical tunnel-support relation proposed by Barton et al. [27], consistent with the reinforcement guidance in USACE [28] and the AASHTO tunnel manual [29]. The minimum length L_{min} depends on tunnel span B and the excavation support ratio ESR :

$$L_{min} = 2 + 0.15 \times \frac{B}{ESR} \text{ (m)}$$

3

where B is the tunnel span and ESR is the excavation support ratio, taken as 1.0 for a permanent railway tunnel.

For the severe-deformation scenarios, the study also considers prestressed cable anchors based on SE-W/SE-N-type technology [30]. These systems use double corrosion protection with HDPE sheathing and can apply active confinement over a longer anchorage length.

2.2. Equivalent Mohr-Coulomb parameters

Although the degraded rock mass is described by the nonlinear Hoek-Brown criterion, the closed-form expressions used for plastic-zone thickness (Eq. 6) and wall convergence (Eq. 9) require equivalent Mohr-Coulomb parameters. The generalized Hoek-Brown parameters m_b , s , and a are therefore converted to equivalent cohesion c and friction angle φ using the stress-dependent fitting procedure of Hoek et al. [5]. The fitting minimizes the difference between the two failure envelopes over the effective confinement range $0 < \sigma_3 < \sigma_{3 \max}$. For deep-tunnel applications, the upper confinement limit is related to rock-mass strength σ_{cm} and far-field effective stress σ_0 as follows:

$$\frac{\sigma_{3 \max}}{\sigma_{cm}} = 0.47 \left(\frac{\sigma_{cm}}{\sigma_0} \right)^{-0.94}$$

4

where σ_{cm} is the global rock-mass compressive strength (MPa)

At the reference overburden $H = 63.77$ m, the effective far-field stress is approximately $\sigma_0 = 1.64$ MPa. The resulting equivalent c and φ values are recalculated across the RMR spectrum and are used as deterministic strength inputs in the subsequent CCM analyses. Because the fitted c and φ values depend on the effective far-field stress (σ_0), they are depth-specific. The subsequent sensitivity analysis of these strength parameters is therefore bounded to the reference overburden.

2.3. Convergence-Confinement Method

To evaluate the mechanical failure of support components and optimize early-stage costs, this analytical framework applies a time-independent, isotropic, and dry continuum model. Heavily fractured rock masses in highly degraded fault zones ($GSI \approx 20$) function as isotropic continua [26]. While true squeezing behavior involves time-dependent creep, groundwater seepage, and anisotropic extrusion [31, 32], coupled modeling requires site-specific 3D hydro-mechanical data typically unavailable during feasibility studies. We bypass this limitation by capturing 3D face extrusion and construction delays indirectly through the dynamic stress-release factor (λ). This constraint restricts the model to preliminary screening rather than final detailed design.

The ground-reaction curve (GRC) is derived for an elastoplastic medium surrounding a circular equivalent tunnel. A hydrostatic far-field stress state is assumed ($K_0 = 1$). Although tectonic fault zones frequently exhibit anisotropic in-situ stresses ($K_0 \neq 1$), the hydrostatic assumption is retained to utilize the 1D closed-form solution. This constraint is acceptable for preliminary support screening. The response is evaluated in two stages: elastic behavior before yielding and elastoplastic behavior after a plastic annulus develops.

When the internal pressure p_i is sufficient to maintain elastic behavior, the radial and tangential stresses are given by the Lamé solution [33]:

$$\sigma_r = \sigma_0 \times \left(1 - \frac{R_0^2}{r^2}\right) + p_i \times \left(\frac{R_0^2}{r^2}\right)$$

5a

$$\sigma_\theta = \sigma_0 \times \left(1 + \frac{R_0^2}{r^2}\right) - p_i \times \left(\frac{R_0^2}{r^2}\right)$$

5b

where σ_0 is the uniform far-field stress, R_0 is the tunnel radius, r is the radial distance from the tunnel center, σ_r is the radial stress, and σ_θ is the tangential stress.

When p_i falls below the elastic limit, a plastic zone develops. For a circular tunnel in a hydrostatic stress field and a Mohr-Coulomb material, the plastic-zone radius R_p is written as [5, 6]:

$$R_p = R_0 \times \left[\frac{2 \times (\sigma_0 (\xi - 1) + q_u)}{(\xi + 1) (p_i (\xi - 1) + q_u)} \right]^{\frac{1}{\xi - 1}}$$

6

where

$$\xi = \frac{1 + \sin\varphi}{1 - \sin\varphi}$$

7

$$q_u = \frac{2c \times \cos\varphi}{1 - \sin\varphi}$$

8

where q_u is the uniaxial compressive strength of the rock mass and p_i is the radial pressure at the tunnel wall.

The radial displacement u at the tunnel wall ($r = R_0$) in the plastic regime is

$$u = R_0 \times \frac{1 + \nu}{E_M} \times \left[2 \times (1 - \nu) \times (\sigma_0 - P_{cr}) \times \left(\frac{R_p}{R_0} \right)^2 - (1 - 2\nu) \times (\sigma_0 - p_i) \right]$$

9

where P_{cr} is the critical support pressure at the elastic-plastic boundary and ν is the Poisson's ratio of the rock mass.

Equation (9) defines the nonlinear relation between internal support pressure p_i and wall displacement u and therefore forms the ground-reaction curve.

2.4. Support characteristic curve

The support characteristic curve (SCC) relates mobilized support pressure to radial displacement. Its slope is governed by the stiffness of the support components. In the present model, the initial support comprises shotcrete with stiffness K_{sc} , while the permanent cast-in-place lining has stiffness K_c .

The shotcrete lining is represented as a thin circular shell. Its radial stiffness is

$$K_{sc} = \frac{E_{sc} \times h_{sc}}{R_0 \times (1 - \nu_{sc}^2)}$$

10

where E_{sc} is the shotcrete elastic modulus, h_{sc} is the shotcrete thickness, and ν_{sc} is the shotcrete Poisson ratio.

The permanent lining is installed after part of the ground deformation has occurred and is represented as a thick-walled cylinder subjected to external pressure. Its radial stiffness is:

$$K_c = \frac{E_c \times (R_{out}^2 - R_{in}^2)}{R_{out} \times (1 - \nu_c) \times [(1 - 2\nu_c) \times R_{out}^2 + R_{in}^2]}$$

11

where E_c is the concrete elastic modulus, R_{out} and R_{in} are the outer and inner lining radii, respectively, and ν_c is the concrete Poisson ratio.

2.5. Integrated techno-economic model

A normalized cost-unit (CU) framework is used to compare support alternatives while limiting the effect of currency fluctuations and regional inflation. The reference price P_{base} is the unit market price of D25 rock-bolt steel. The total direct support cost per tunnel meter, C_{total} , is calculated by summing the normalized material, labor, and equipment components:

$$C_{total} (CU) = \sum (Q_{Mi} \times K_{Mi}) + \sum (Q_{Li} \times K_{Li}) + \sum (Q_{Ei} \times K_{Ei})$$

12

where Q_i denotes the relevant consumption quantity (e.g., kg/m or shift/m) and K_i is the price coefficient normalized by the reference steel price, $K = Price_i / Price_{steel}$. By setting 1 kg of D25 steel as the baseline cost unit (1 kg steel $\equiv$ 1 CU), K_i takes the dimension of CU per physical unit (e.g., CU/kg or CU/shift). This substitution ensures that the aggregated cost C_{total} maintains a consistent dimension of CU/m. The Q and K values are calibrated using Vietnamese construction norms and market databases for underground works [20–22, 34–37]. The framework therefore compares the relative direct cost of alternative support strategies on a consistent local basis. Although calibrated with local datasets, this normalized CU framework and its resulting risk-cost matrix (as developed in Section 4) can be directly applied to other international tunneling projects by substituting the regional resource database.

Supplementary Table S1 lists the conversion factors for the principal resources. Specialized tunneling equipment has the largest conversion factors, illustrating the capital intensity of the NATM cycle. At the time of the analysis, 1 CU corresponded approximately to the standardized price of 1 kg of D25 rock-bolt steel; the indicative monetary conversion is reported only for transparency and is not used as the optimization variable.

Although absolute prices vary with location and time, the adopted consumption rates and relative price coefficients provide a consistent basis for comparing conventional NATM support options.

Supplementary Tables S1 and S2 summarize the normalized cost and resource inputs used in the optimization.

To implement Eq. (12), each support activity is decomposed into its principal material, labor, and equipment inputs. Supplementary Table S2 reports the standardized consumption coefficients derived from the national construction norms [20].

3. CASE STUDY AND ANALYSIS SCENARIOS

The representative alignment crosses the Truong Son fold belt, where sandstone and siltstone are interbedded with shale and may be affected by weathering and tectonic disturbance [2, 3]. At the reference section, the overburden is $H = 63.77$ m. The circular-equivalent CCM model provides a baseline approximation. This 1D simplification omits local stress concentrations at the invert and corners of the actual horseshoe profile, restricting the outputs to average radial deformations.

JICA investigations in the central geological belt indicate that the principal design uncertainty is the spatial heterogeneity of the rock mass rather than overburden alone [3]. Weathered interbeds and minor fault zones can locally reduce RMR from the baseline fair-rock value of 59 to poor-rock conditions below 45. The sensitivity analyses are designed to quantify the resulting change in deformation and support demand.

3.1. Geometry and baseline geotechnical data

The case-study tunnel has a horseshoe cross-section compatible with a 1435 mm high-speed railway clearance gauge and a design speed above 200 km/h. The representative geological unit is weathered sandstone with a baseline RMR of 59. Figure 1a and Table 1 summarize the tunnel geometry, baseline geotechnical parameters and support properties.

3.2. Support-system parameters

The baseline primary support consists of 150 mm shotcrete and systematic rock bolts in accordance with the adopted NATM design. Table 1 also lists the mechanical properties used in the CCM analysis.

3.3. Representative calculation

The calculation procedure is illustrated for the baseline case ($RMR = 59$, $H = 63.77$ m). The CCM formulation is summarized conceptually in Fig. 1b, following the framework described by Oreste [38], and the numerical ground-response results are shown in Fig. 2.

The excavation response is evaluated using the closed-form CCM solution with a Mohr-Coulomb yield criterion [5, 6]. To avoid overestimating rock-mass stiffness in discontinuous ground, EM is obtained from the adopted RMR-based relation. For the baseline case ($RMR = 59$, $\varphi = 33^\circ$), Fig. 2 shows the evolution of wall displacement u and plastic-zone thickness t_p as the fictitious internal pressure p_i decreases below the elastic limit.

In Fig. 2a, the elastic limit is $P_{cr} = 0.54$ MPa and the calculated unsupported wall displacement is $u_{max} = 1.17$ mm. Below P_{cr} , displacement increases nonlinearly as confinement is released.

Figure 2b reports the plastic-zone thickness $t_p = R_p - R_0$ rather than only the absolute plastic radius. At zero internal pressure, the calculated yielded annulus extends 2.63 m from the excavation boundary. This measure can be compared directly with the length of bolt embedded beyond the yielded zone and is therefore used in the subsequent anchorage assessment.

3.4. Analysis scenarios

Four scenarios are defined to represent uncertainty in rock quality, overburden, support capacity, and material parameters. The variables and ranges are summarized in Supplementary Table S3.

Scenario 1 varies RMR from 30 to 80 to quantify the nonlinear development of the plastic-zone thickness and to compare it with the available bolt embedment.

Scenario 2 varies overburden from 20 to 150 m at the baseline RMR. The resulting equilibrium pressure and plastic-zone thickness are used to evaluate the shotcrete capacity and the stability of the bolt anchorage.

Scenario 3 progressively reduces RMR while retaining the baseline 150 mm M30 shotcrete and the existing bolt system. The purpose is to identify the model-predicted support threshold and the component that controls the loss of system performance.

Scenario 4 performs a local sensitivity analysis of friction angle ϕ , cohesion c , and overburden H around the degraded base case. The perturbation ranges used for each parameter are reported in Fig. 4b and Supplementary Table S3, and the response variable is the plastic-zone thickness t_p .

3.5. Use of generative AI tools

OpenAI ChatGPT was used for language editing, consistency checks and manuscript formatting. All analytical formulations, calculations, data interpretation, figures and scientific conclusions were reviewed and verified by the authors, who take full responsibility for the content of the manuscript.

4. RESULTS AND DISCUSSION

4.1. Effect of rock-mass quality

Scenario 1 shows a strongly nonlinear increase in plastic-zone thickness as RMR decreases (Fig. 3a). Using t_p rather than the absolute radius R_p provides a direct geometric comparison with the available bolt embedment. The curve steepens in the poor-rock range, indicating that relatively small reductions in rock-mass quality can produce disproportionately large increases in the yielded annulus.

For RMR values above 60, yielding remains comparatively localized. As rock quality deteriorates, the reduction in frictional resistance causes the plastic annulus to expand rapidly. The nominal bolt length is 3.95 m; after reserving 1.20 m of anchorage in competent rock, the available effective length is $L_{eff} = 2.75 \text{ m}$.

The model identifies a transition near RMR = 49 at which the available effective embedment is exhausted. Above this threshold, the bolts retain an anchored segment in the elastic rock mass. Below it, only residual grout-rock interface resistance remains, and the bolts cannot be assumed to provide their nominal pull-out capacity [7, 26]. This condition motivates the assessment of longer prestressed cable anchors for the degraded zones.

4.2. Effect of overburden depth

In Scenario 2, equilibrium pressure increases approximately with overburden, whereas the plastic-zone thickness remains below the available bolt embedment over the investigated range (Fig. 3b). The response is evaluated against the nominal shotcrete capacity and the 2.75 m effective bolt length.

At $H = 150 \text{ m}$, the calculated equilibrium pressure reaches 1.02 MPa, corresponding to approximately 88% of the assumed 1.16 MPa shotcrete capacity. The remaining margin is therefore limited and may be reduced further by time-dependent deformation or nonuniform loading not represented in the axisymmetric model.

At the same depth, the predicted plastic-zone thickness is approximately 0.84 m, which remains below the 2.75 m effective bolt length. The model therefore indicates that shotcrete demand, rather than bolt pull-out, governs the depth-only scenario at the baseline rock quality.

These results show that a 150 mm shotcrete lining should not be interpreted as excessive solely on the basis of average RMR. At the largest modeled overburden, it provides a limited reserve before invert closure and installation of the permanent lining.

4.3. Component-specific safety limits

Within the axisymmetric CCM idealization, the term asynchronous failure refers to unequal mobilization and loss of capacity among support components rather than to spatially asymmetric convergence. As rock quality deteriorates, the deformation-controlled bolt system may lose stable embedment while the shotcrete remains below its compressive limit [26, 39, 40]. For the depth scenario, an equilibrium pressure of 1.02 MPa corresponds to approximately 87.9% of the assumed shotcrete capacity, giving a limited nominal safety margin.

A single system-level safety factor can therefore obscure the controlling mode. The dashboard in Fig. 4a compares shotcrete pressure demand with the pre-support plastic-zone thickness and the effective bolt length. This dual criterion distinguishes stress demand on the lining from the kinematic loss of bolt anchorage and identifies the support component that controls the predicted transition.

An analysis based on fully mobilized shotcrete confinement may predict stability at very low RMR values, but that result assumes instantaneous support activation. The construction sequence necessarily permits some stress release and convergence before the lining becomes effective. The pre-support response must therefore be evaluated separately.

During the unsupported interval, the unexcavated core provides a three-dimensional face effect that can be represented in the one-dimensional CCM by a fictitious internal pressure. A bounded stress-release factor $\lambda = 0.15 - 0.255$ is used here to represent early and delayed support installation, respectively [4, 5, 41]. Across this range, the model places the bolt-embedment transition between RMR = 48.2 and 50.1. The threshold is thus better interpreted as an operational band centered near RMR = 49 than as a unique material constant. Once the plastic annulus consumes the available embedment, the 3.95 m bolts can no longer be assigned their nominal pull-out resistance. This loss of anchorage induces bolt slip, forcing stress redistribution onto the shotcrete. This load transfer can trigger lining failure before the nominal 1.16 MPa compressive limit is reached.

Accordingly, the RMR = 48.2–50.1 interval is used as a model-specific intervention band. For poorer ground, the analysis indicates that reliance on the baseline short passive-bolt pattern requires explicit verification or replacement by a system capable of anchoring beyond the yielded annulus.

4.4. Sensitivity analysis

The local sensitivity analysis in Scenario 4 evaluates the change in t_p caused by perturbations in φ , c , and H (Fig. 4b). Within the investigated ranges, the strength parameters have a larger influence on the yielded-zone thickness than overburden.

Overburden has the smallest local effect, with a reported change in t_p of approximately 0.17 m. A 20% reduction in cohesion increases the plastic-zone thickness by about 1.1 m, indicating the importance of intact rock bridges and cementation within the disturbed mass.

The friction angle is the most influential parameter in the adopted perturbation range. A 10% reduction from 33.0° to 29.7° increases t_p to 3.34 m, exceeding the 2.75 m effective bolt length. The result emphasizes the need to characterize residual friction along weakness planes in faulted or highly fractured ground rather than relying only on average overburden.

4.5. Rock-support interaction at the critical state

A single deterministic pre-convergence factor is unlikely to represent construction in severely degraded ground (RMR = 40). Longitudinal displacement profile formulations relate unsupported length to radial convergence, but their application to fractured fault zones is affected by face extrusion, overbreak, equipment interruptions, and variable advance rate [5, 41]. The analysis therefore uses $\lambda = 0.15$ for early support installation and $\lambda = 0.255$ for delayed installation to define a bounded operational response before shotcrete activation.

Figure 5. Critical-state interaction and support optimization. (a) Ground-reaction/support-characteristic response and plastic-zone thickness relative to effective bolt embedment for the bounded installation range at RMR = 40. (b) Cost versus effective anchorage margin for the four support strategies at RMR = 35.

Figure 5a compares the ground-reaction curve with the family of support-characteristic curves associated with the adopted installation range. Rather than a single equilibrium point, the construction sequence produces an equilibrium band. This representation is used to assess whether the convergence required to mobilize shotcrete also exhausts the effective bolt embedment. As calculated in Fig. 5a, the equilibrium plastic zone thickness spans from 1.76 m to 3.3 m, physically breaching the 2.75 m effective embedment limit of the baseline bolts.

When the yielded annulus extends beyond the effective embedment, the passive bolts retain only limited interface resistance and cannot be credited with their nominal global confinement. Under the severe fault-zone scenario, the baseline system should therefore be replaced by 8–10 m prestressed cable anchors. These deep active anchors provide sufficient embedment length and yielding capacity to accommodate large radial convergence without premature pull-out failure. This recommendation is conditional on verification of the component thresholds shown in Fig. 5a.

4.6. Techno-economic optimization of support strategies

The cost optimization is performed for a severe scenario with RMR = 35. The calculated unsupported plastic radius is $R_p = 14.5$ m, corresponding to a yielded-zone thickness far greater than the available embedment of the baseline 3.95 m bolts. The objective is to satisfy a target safety factor of 1.2 at minimum normalized direct cost.

Four strategies are compared: densification of the existing bolts (Strategy A), increased shotcrete thickness (Strategy B), a combined redesign (Strategy C), and deep prestressed cable anchorage (Strategy D). The comparison is expressed in the CU framework defined in Section 2.5.

The optimization requires a nonnegative anchorage margin after applying the target factor of safety to the plastic-zone thickness. The criterion is

$$M_{Safety} = L_{eff} - 1.2t_p$$

13

where equilibrium satisfies $M_{Safety} \geq 0$.

Figure 5b shows that the baseline design has a negative anchorage margin under the severe scenario. The optimization therefore evaluates whether additional expenditure produces a mechanically meaningful increase in embedment.

Increasing shotcrete thickness does not substantially reduce the deep-seated yielded-zone thickness and remains in the unsafe region over the investigated range. Densifying the 3.95 m bolts also fails to satisfy the geometric anchorage criterion, even at the minimum operational spacing considered. These strategies increase cost without extending reinforcement beyond the plastic annulus.

Among the modeled options, deep prestressed anchorage provides the lowest cost at the required safety margin. The continuous solution gives an anchor length of 9.61 m and a normalized direct cost of 2909 CU/m. This result should be interpreted as the optimum within the adopted analytical and cost assumptions rather than as a universal design value [42, 43].

A rigid 10 m bar would be difficult to handle within the available tunnel-face clearance. Flexible cable tendons can instead be fed into boreholes drilled with short-feed equipment and extension rods. Long cable anchors have also been used in severe squeezing conditions, including the Driskos case discussed in Section 4.7 [26, 31].

Field implementation requires discrete tendon lengths and includes a prescribed allowance for material loss. Rounding the continuous optimum of 9.61 m to a 10 m tendon and applying the prescribed 12.5% statutory wastage allowance exclusively to the tendon material component increases the estimated total as-built cost to 3105.4 CU/m (representing a 6.75% net increase over the theoretical continuous optimum). The calculation therefore distinguishes the mathematical optimum from the field-feasible support package.

Figure 6. Risk-cost response and case-history back-analysis. (a) Bivariate risk-cost map for rock-mass quality and support-installation delay. (b) Calculated longitudinal displacement profile compared with published monitoring data for the Driskos Twin Tunnel; field monitoring data were digitized from Vlachopoulos et al. [32].

For competent rock ($RMR > 60$), the isocost contours are widely spaced and the modeled cost is relatively insensitive to λ . Near the model-specific embedment transition around $RMR = 49$, the contours become closely spaced, indicating a sharp cost gradient.

Across $\lambda = 0.15 - 0.255$ at the transition, the calculated direct support cost ranges from 2948 to 3698 CU/m, with the field-adjusted reference case near 3105 CU/m. For $RMR < 45$, the cost can exceed the adopted non-viability threshold of 4500 CU/m. This escalation occurs because additional short passive bolts do not recover stable embedment, whereas deeper active anchorage directly addresses the governing geometric limitation [44].

4.7. Back-analysis of a published case history

Because excavation data are not yet available for the North-South HSR project, the analytical framework is compared with the published Driskos Twin Tunnel case on the Egnatia Highway in Greece. The 12 m span tunnel crosses highly degraded siltstone flysch with a mean GSI of approximately 20, intact strength near 20 MPa, and cover up to 220 m [26, 31]. These conditions provide a useful, although not

identical, benchmark for the severe end of the present parametric study. Table 2 summarizes the comparison.

Table 2
Geological and geometric comparison used for the Driskos back-analysis.

Parameter	North-South HSR (worst-case envelope)	Driskos Twin Tunnel (chainage 8300–9000)
Tunnel span	13.0 m	12.0 m
Cover depth	63.77–294.0 m	Up to 220 m
Initial in-situ stress	1.64–7.80 MPa	5.72 MPa
Rock-mass quality	RMR = 59 (baseline), degrading to 30	GSI = 20 (mean siltstone flysch)
Intact compressive strength	25 MPa	20 MPa (mean)

Published accounts of the Driskos fault zone report that passive bolts and shotcrete were insufficient to preserve the required clearance and that additional tensioned cables were installed [26]. The case therefore supports the physical plausibility of transitioning to longer active anchorage when the yielded zone exceeds the effective length of passive bolts.

Figure 6b compares the calculated longitudinal displacement profile with published Driskos monitoring data. The field dataset was explicitly extracted from Vlachopoulos et al. [32] using optical digitization tools. The analytical curve captures the overall trend of the measurements but intrinsically underestimates convergence near the face ($x \approx 0$), where three-dimensional spatial extrusion is not represented by the one-dimensional CCM formulation. The bounded stress-release factor ($\lambda = 0.15 - 0.255$) adopted in the predictive model is explicitly introduced to compensate for this 1D limitation by enveloping the unmodeled 3D face extrusion prior to support activation. However, in the far-field domain (where the 1D plane-strain assumption is rigorously valid), the predicted final convergence plateau of 160.0 mm aligns closely with the stabilized field measurement mean of 147.9 mm, yielding a highly acceptable relative error of under 8.2%. Recognizing the theoretical limitations near the active face, this comparison should therefore be interpreted as a robust semi-quantitative back-analysis rather than a full statistical validation.

5. CONCLUSIONS

The study integrates the Convergence-Confinement Method with a normalized cost model to examine component-specific support response in squeezing ground. Within the adopted assumptions, degradation of rock-mass quality can exhaust the effective embedment of 3.95 m passive bolts before the 150 mm shotcrete lining reaches its nominal capacity. The transition is predicted over an RMR band of approximately 48.2–50.1 when support-installation variability is represented by $\lambda = 0.15 - 0.255$.

For the severe RMR = 35 scenario, the continuous cost optimum requires a 9.61 m anchorage length; a field-standard 10-m prestressed cable tendon, after accounting for material wastage allowances, gives an estimated as-built cost of 3105.4 CU/m.

The framework is intended for preliminary screening and comparison of support strategies, not for final design. Its principal limitations are the circular, axisymmetric, and time-independent CCM assumptions; the simplified representation of construction delay; and the omission of groundwater, anisotropy, and three-dimensional face behavior. Future work should employ 3D coupled hydro-mechanical modeling, incorporating time-dependent creep and groundwater seepage, to verify the RMR = 48.2–50.1 intervention band proposed in this preliminary study.

Declarations

Funding

This research was supported by the Ministry of Education and Training of Vietnam under Grant CTB2025-GHA-06.

Author contributions

Huy Nghia Nguyen: Methodology, Formal analysis, Software, Validation, Investigation, Data curation, Writing - original draft, Writing - review and editing. The Truyen Tran: Conceptualization, Methodology, Formal analysis, Supervision, Writing - review and editing. Xuan Tung Nguyen: Software, Validation, Writing - review and editing. Minh Ngoc Vu: Software, Visualization, Writing - review and editing.

Competing interests

The authors declare no competing interests.

Data availability

The data and analytical models supporting the findings of this study are available from the corresponding author upon reasonable request. The digitized Driskos Twin Tunnel case-history data used for the back-analysis, together with the data-extraction procedure, are also available upon reasonable request. The original monitoring data were obtained from the published source cited in the manuscript.

References

1. Bieniawski, Z. T. *Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering* (Wiley, 1989).
2. Ministry of Transport of Vietnam. Pre-Feasibility Study Report: North-South High Speed Railway Project. Hanoi. (2024).

3. Japan International Cooperation Agency (JICA), Vietnam Railways (VR). Study for the Formulation of High Speed Railway Projects on Hanoi-Vinh and Ho Chi Minh-Nha Trang Section. Hanoi. (2013).
4. Panet, M. Calculation of Tunnels by the Convergence-Confinement Method Press of the Ecole Nationale des Ponts et Chaussees, Paris, (1995).
5. Hoek, E., Carranza-Torres, C. & Corkum, B. Hoek-Brown failure criterion-2002 edition. In: Proceedings of NARMS-TAC, vol. 1, pp. 267–273 (2002).
6. Carranza-Torres, C. & Fairhurst, C. Application of the convergence-confinement method of tunnel design to rock masses that satisfy the Hoek-Brown failure criterion. *Tunn. Undergr. Space Technol.* **15** (2), 187–213. [https://doi.org/10.1016/S0886-7798\(00\)00046-8](https://doi.org/10.1016/S0886-7798(00)00046-8) (2000).
7. Kontogianni, V. A. & Stiros, S. C. Induced deformation during tunnel excavation: evidence from geodetic monitoring. *Eng. Geol.* **79** (1–2), 115–126. <https://doi.org/10.1016/j.enggeo.2004.10.012> (2005).
8. Chang, L., Alejano, L. R., Cui, L., Sheng, Q. & Xie, M. Limitation of convergence-confinement method on three-dimensional tunnelling effect. *Sci. Rep.* **13** (1), 1988. <https://doi.org/10.1038/s41598-023-29062-5> (2023).
9. Pu, S. et al. Analysis of deformation characteristics and failure mechanism of interbedded surrounding rock tunnels based on principal stress difference. *Int. J. Geomech.* **24** (9), 04024197. <https://doi.org/10.1061/IJGNAL.GMENG-9558> (2024).
10. Sun, Q. et al. Study on the asymmetric failure characteristics and failure mechanisms of surrounding rock during excavation of a deep buried small-clearance tunnel. *Appl. Sci.* **15** (9), 4763. <https://doi.org/10.3390/app15094763> (2025).
11. Zhang, Y. & Zeng, Y. Symmetric and asymmetric failure mechanism of deep tunnel excavation under complex environmental conditions. *Symmetry* **17** (3), 416. <https://doi.org/10.3390/sym17030416> (2025).
12. Do, N. A., Dias, D., Oreste, P. & Djeran-Maigre I. 2D numerical investigations of twin tunnel interaction. *Geomech. Eng.* **6** (3), 263–275. <https://doi.org/10.12989/gae.2014.6.3.263> (2014).
13. Hoek, E. & Guevara, R. Overcoming squeezing in the Yacambu-Quibor Tunnel, Venezuela. *Rock. Mech. Rock. Eng.* **42** (2), 389–418. <https://doi.org/10.1007/s00603-009-0175-5> (2009).
14. Ding, K., Wang, L., Ren, B., Wang, S. & Jiang, C. Study on optimization of initial support for a tunnel in the fracture zone based on the strength reduction method. *Processes* **10** (8), 1558. <https://doi.org/10.3390/pr10081558> (2022).
15. Ekmen, A. B. & Avci, Y. Artificial intelligence-assisted optimization of tunnel support systems based on multiple three-dimensional finite element analyses considering the excavation stages. *Iran. J. Sci. Technol. Trans. Civ. Eng.* **47** (3), 1725–1747. <https://doi.org/10.1007/s40996-023-01109-7> (2023).
16. Guo, F. Multi-objective optimization of tunnel displacement control construction parameters based on nonlinearity. In: Proceedings of the 2024 3rd International Conference on Structural Seismic Resistance, Monitoring and Detection (SSRMD 2024), p. 377. Springer Nature (2024).

17. Ren, H., Zhu, Y., Yao, Q., Li, P. & Wei, M. Study on the mechanism of anchoring parameters of anchor rods in cemented broken surrounding rock. *Sci. Rep.* **15** (1), 28256. <https://doi.org/10.1038/s41598-025-13442-0> (2025).
18. Carranza-Torres, C. & Hormazabal, E. Computational tools for the estimation of factor of safety and location of the critical failure surface for slopes in rock masses that satisfy the Hoek-Brown failure criterion. In: *Slope Stability 2020: Proceedings of the 2020 International Symposium on Slope Stability in Open Pit Mining and Civil Engineering*, pp. 1099–1122. Australian Centre for Geomechanics, Perth (2020).
19. Li, D. A new analytical model for stress distribution in the rock bolt under axial loading. *Int. J. Rock. Mech. Min. Sci.* **176**, 105690. <https://doi.org/10.1016/j.ijrmms.2024.105690> (2024).
20. Ministry of Construction. *Circular No. 12/2021/TT-BXD: Promulgating Construction Norms* (Hanoi, 2021).
21. People's Committee of Thua Thien Hue Province. Decision No. 1176/QĐ-UBND, Appendix 3: Classification of Labor Groups for Construction Works. (2025a).
22. Thua Thien Hue Department of Construction. Announcement of Construction Material Prices in Thua Thien Hue Province. (2025).
23. Hoek, E. & Diederichs, M. S. Empirical estimation of rock mass modulus. *Int. J. Rock. Mech. Min. Sci.* **43** (2), 203–215. <https://doi.org/10.1016/j.ijrmms.2005.06.005> (2006).
24. Serafim, J. L. & Pereira, J. P. Considerations on the geomechanics classification of Bieniawski. In: *Proceedings of the International Symposium on Eng. Geol. and Underground Construction*, vol. 1, pp. 31–42 (1983).
25. Lowson, A. R. & Bieniawski, Z. T. Critical assessment of RMR-based tunnel design practices: a practical engineer's approach. In: *Rapid Excavation and Tunneling Conference 2013 Proceedings*, p. 180. SME (2013).
26. Hoek, E. & Brown, E. T. The Hoek-Brown failure criterion and GSI-2018 edition. *J. Rock. Mech. Geotech. Eng.* **11** (3), 445–463. <https://doi.org/10.1016/j.jrmge.2018.08.001> (2019).
27. Barton, N., Lien, R. & Lunde, J. Engineering classification of rock masses for the design of tunnel support. *Rock. Mech.* **6** (4), 189–236. <https://doi.org/10.1007/BF01239496> (1974).
28. US Army Corps of Engineers (USACE). *Rock Reinforcement. Engineer Manual EM 1110-1-2907* (Department of the Army, 1980).
29. National Highway Institute (US). *Technical Manual for Design and Construction of Road Tunnels-Civil Elements* (AASHTO, 2010).
30. SE Corporation. *Technical Data for SEEE Ground Anchors: SE-W and SE-N Types* (Product Catalog, 2019).
31. Hoek, E. & Marinos, P. Predicting tunnel squeezing problems in weak heterogeneous rock masses. *Tunnels Tunn. Int.* **32** (11), 45–51 (2000).

32. Vlachopoulos, N., Diederichs, M. S., Marinos, V. & Marinos, P. Tunnel behaviour associated with the weak Alpine rock masses of the Driskos Twin Tunnel system, Egnatia Odos Highway. *Can. Geotech. J.* **50** (1), 91–120. <https://doi.org/10.1139/cgj-2012-0025> (2013).
33. Kirsch, E. G. Die Theorie der Elastizitat und die Bedurfnisse der Festigkeitslehre. *Z. Ver. Dtsch. Ing.* **42**, 797–807 (1898).
34. People's Committee of Thua Thien Hue Province. Decision No. 1176/QD-UBND, Appendix 4: Unit Labor Costs for Construction Region II. (2025b).
35. People's Committee of Thua Thien Hue Province. Decision No. 1177/QD-UBND, Appendix 2: List of Construction Machinery and Equipment. (2025c).
36. People's Committee of Thua Thien Hue Province. Decision No. 1177/QD-UBND, Appendix 3: Machinery Shift Unit Prices (Cost Norms). (2025d).
37. People's Committee of Thua Thien Hue Province. Decision No. 1177/QD-UBND, Appendix 4: Specialized Construction Equipment Database. (2025e).
38. Oreste, P. The convergence-confinement method: roles and limits in modern geomechanical tunnel design. *Am. J. Appl. Sci.* **6**, 757–771 (2009).
39. Bobet, A. & Einstein, H. H. Tunnel reinforcement with rockbolts. *Tunn. Undergr. Space Technol.* **26** (1), 100–123. <https://doi.org/10.1016/j.tust.2010.06.006> (2011).
40. Carranza-Torres, C. Elasto-plastic solution of tunnel problems using the generalized form of the Hoek-Brown failure criterion. *Int. J. Rock. Mech. Min. Sci.* **41**, 629–639. <https://doi.org/10.1016/j.ijrmms.2004.03.111> (2004).
41. Vlachopoulos, N. & Diederichs, M. S. Improved longitudinal displacement profiles for convergence confinement analysis of deep tunnels. *Rock. Mech. Rock. Eng.* **42** (2), 131–146. <https://doi.org/10.1007/s00603-009-0176-4> (2009).
42. Hutchinson, D. J. & Diederichs, M. S. Cablebolting in Underground Mines (1996). <https://www.osti.gov/etdeweb/biblio/463937>
43. Windsor, C. R. Rock reinforcement systems. *Int. J. Rock Mech. Min. Sci.* **34** (6), 919–951. [https://doi.org/10.1016/S1365-1609\(97\)80004-4](https://doi.org/10.1016/S1365-1609(97)80004-4) (1997).
44. Komurlu, E. & Demir, S. Use of Rock Mass Rating (RMR) values for support designs of tunnels excavated in soft rocks without squeezing problem. *GeoScience Eng.* **65** (2), 1–17. <https://doi.org/10.35180/gse-2019-0007> (2019).

Figures

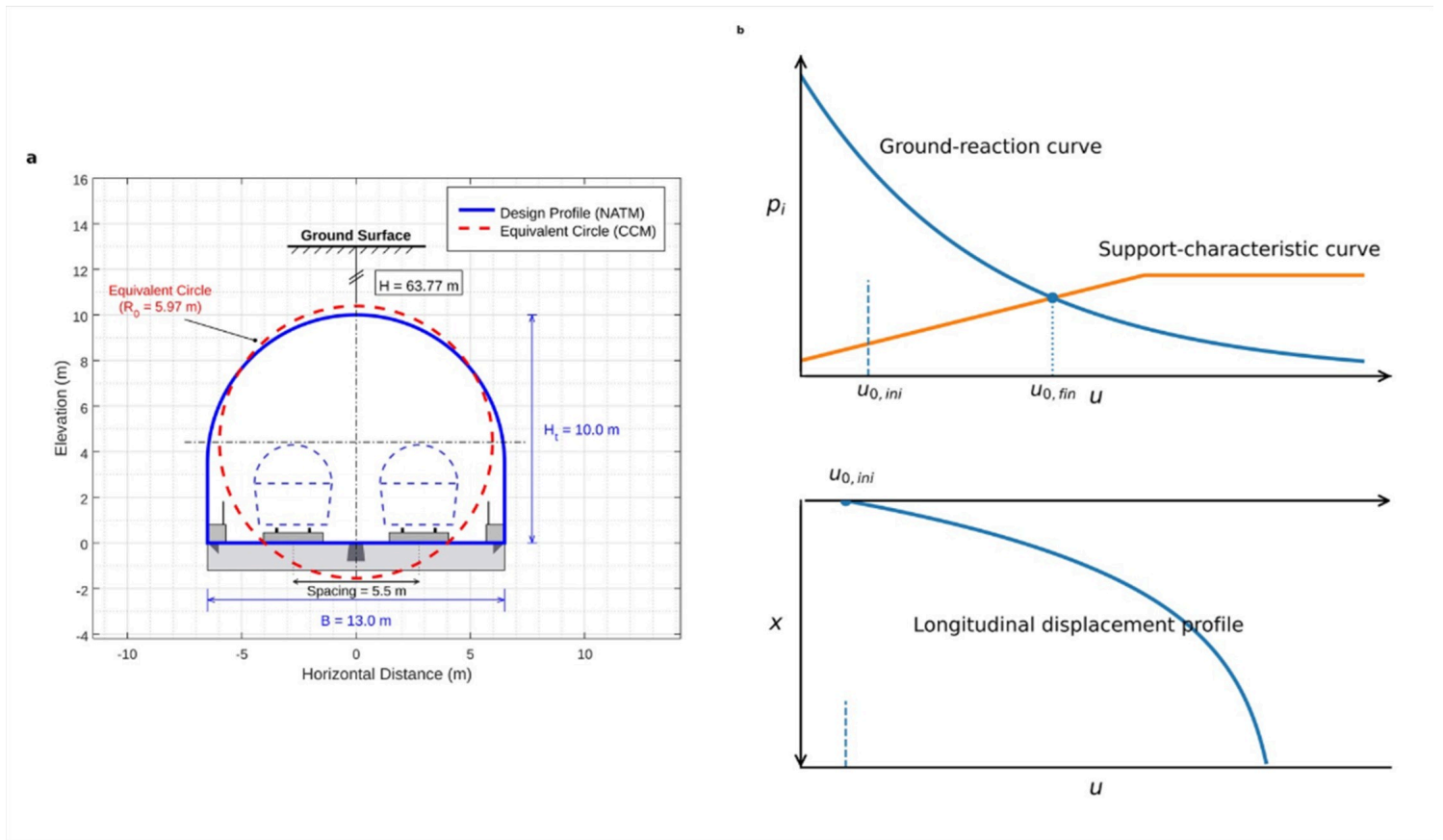


Figure 1

Geometric and conceptual basis of the analytical framework. (a) High-speed railway tunnel cross-section and circular-equivalent geometry used in the CCM analysis. (b) Schematic of the convergence-confinement method showing the ground-reaction curve, support-characteristic curve and longitudinal displacement profile.

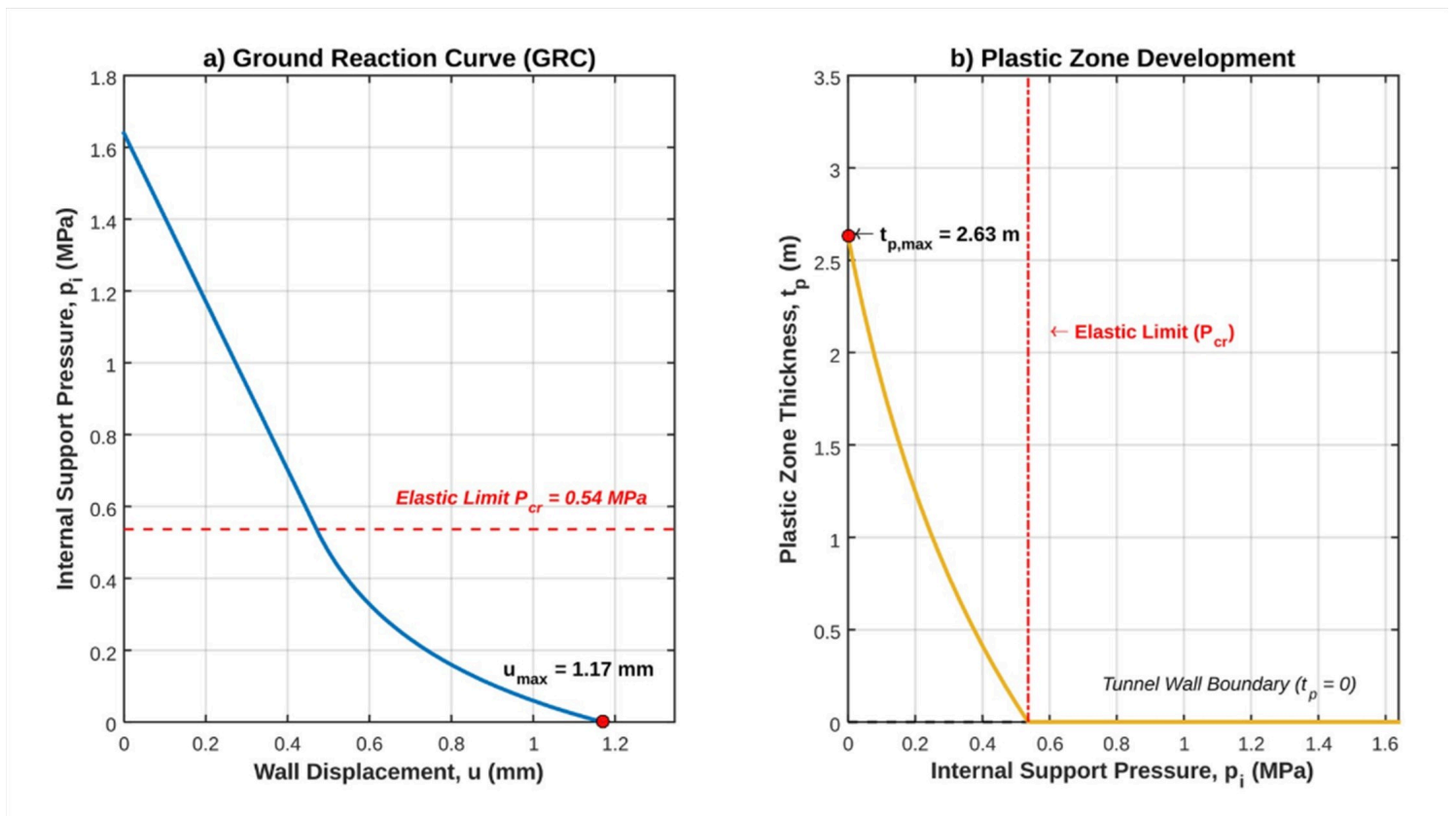


Figure 2

Baseline analytical response for RMR = 59 and H = 63.77 m: (a) ground-reaction curve and (b) plastic-zone development.

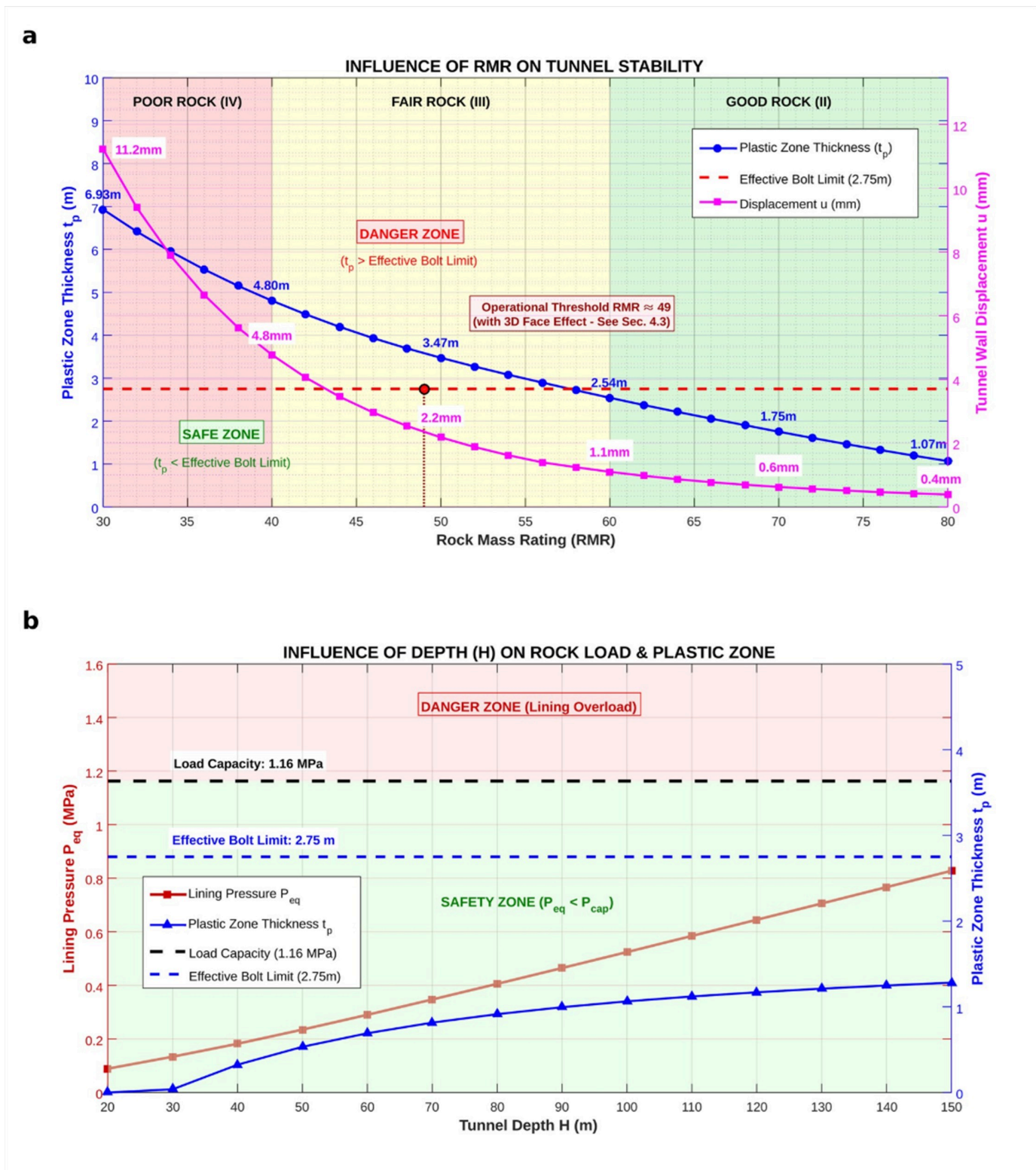


Figure 3

Influence of governing ground conditions on support response. (a) Effect of Rock Mass Rating on plastic-zone thickness and wall displacement relative to the effective bolt-embedment limit. (b) Combined assessment of shotcrete pressure demand and bolt-embedment capacity with increasing overburden.

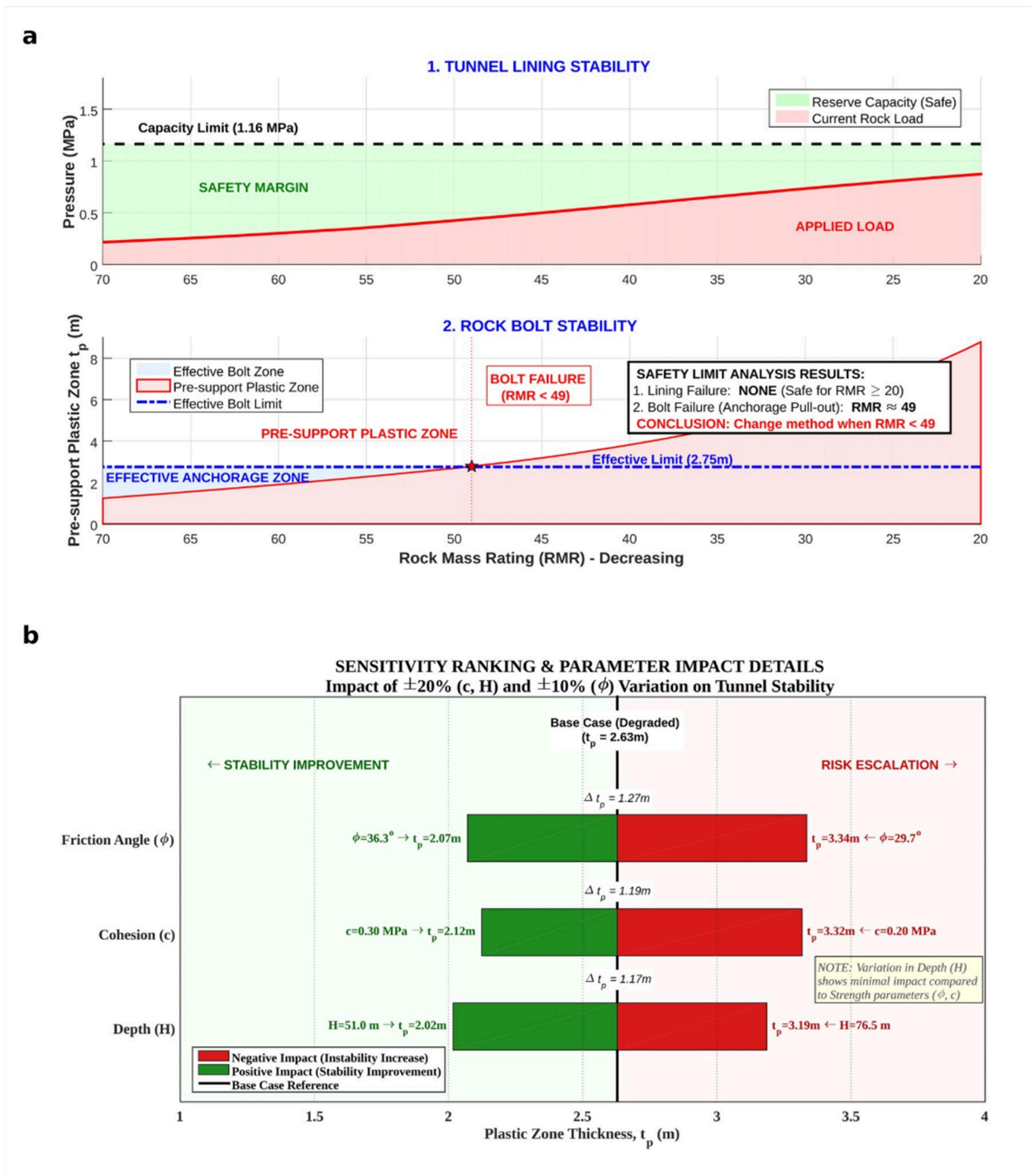


Figure 4

Component-level support limits and parameter sensitivity. (a) Component-specific assessment of shotcrete demand and passive-bolt anchorage. (b) Local sensitivity of plastic-zone thickness to the parameter perturbations adopted for the degraded base case.

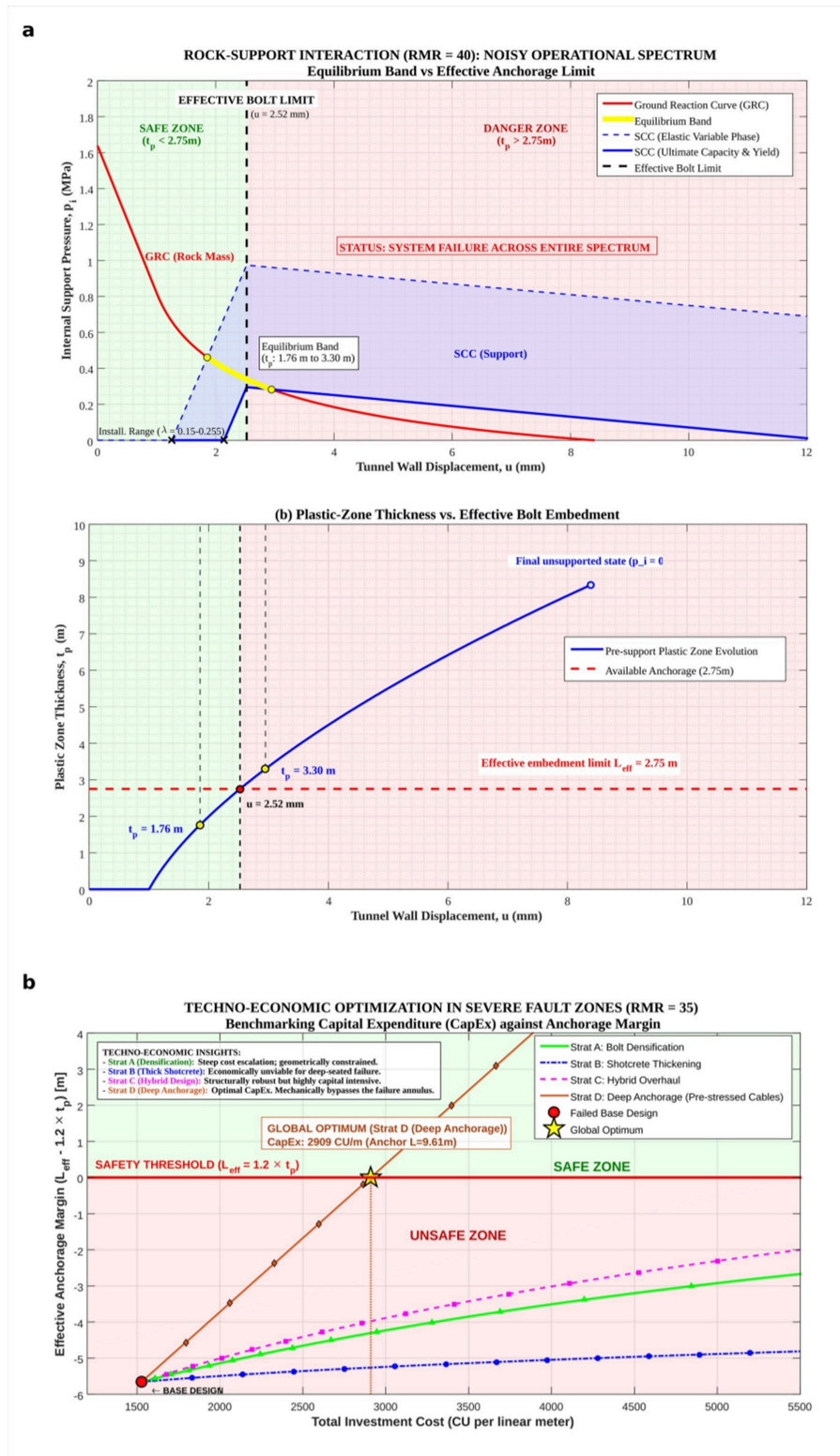


Figure 5

Critical-state interaction and support optimization. (a) Ground-reaction/support-characteristic response and plastic-zone thickness relative to effective bolt embedment for the bounded installation range at RMR = 40. (b) Cost versus effective anchorage margin for the four support strategies at RMR = 35.

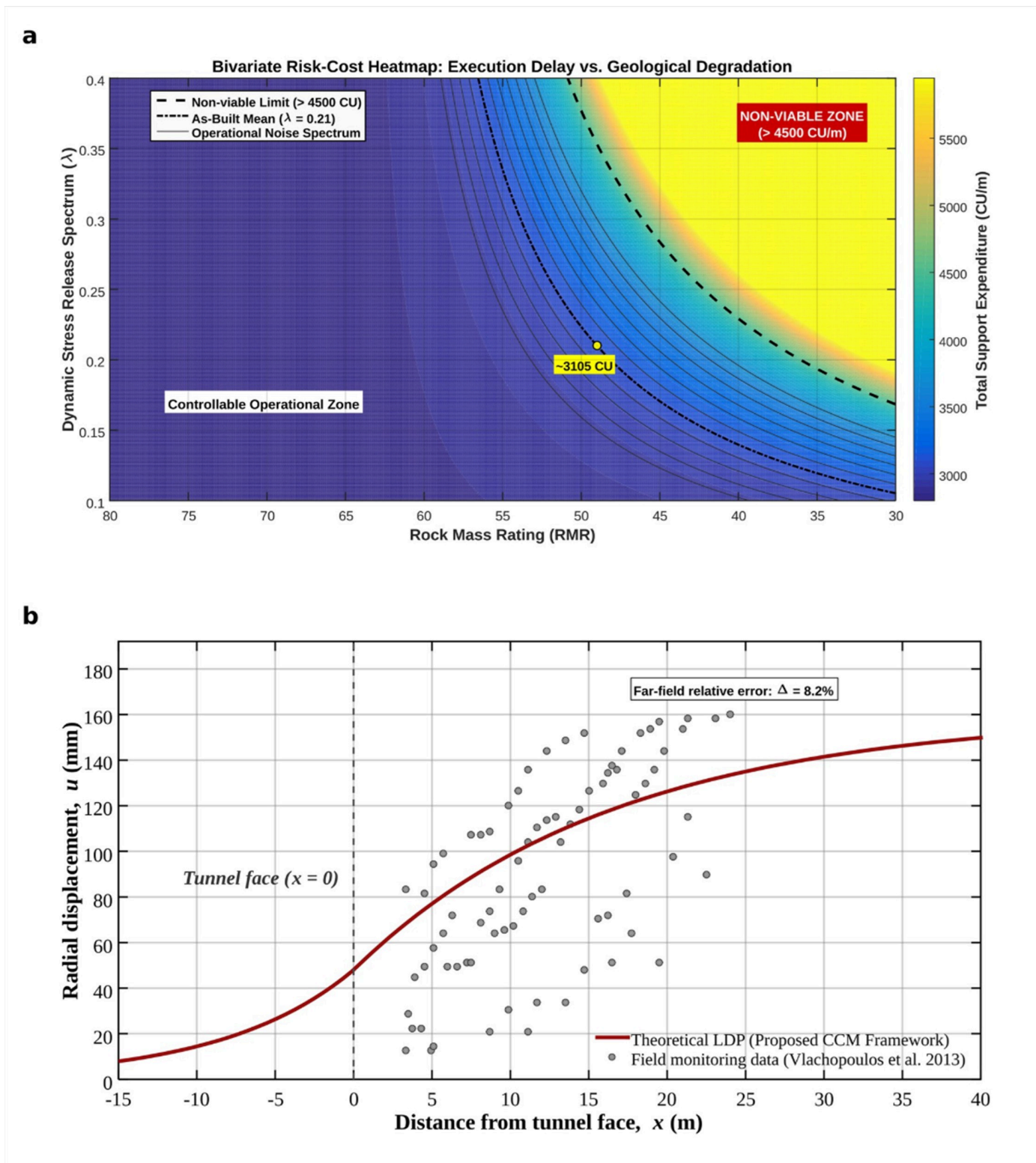


Figure 6

Risk-cost response and case-history back-analysis. (a) Bivariate risk-cost map for rock-mass quality and support-installation delay. (b) Calculated longitudinal displacement profile compared with published monitoring data for the Driskos Twin Tunnel; field monitoring data were digitized from Vlachopoulos et al. [32].