



ARBEITSBERICHT NAB 25-02

Simulation of EDZ Fracture Patterns
Around the HLW- and L/ILW-Emplacement
Rooms of a Deep Geological Repository in
the Siting Region NL (SGT-E3)

January 2025

**Nagra | National Cooperative for the
Disposal of Radioactive Waste**

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Geomechanica Inc., Canada

KEYWORDS

Numerical simulation, EDZ, fracture network, Finite-Discrete
Element Method (FDEM), Opalinus Clay

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1 Introduction

1.1 Background and motivation

The procedure for selecting the sites for the deep geological repositories in Switzerland was specified in the conceptual part of the Sectoral Plan for Deep Geological Repositories (SGT) (BFE 2008) and consisted of three stages. In Stage 3, the three remaining potential siting regions Jura Ost (JO), Nördlich Lägern (NL) and Zürich Nordost (ZNO) were examined in detail and, where necessary, supplementary geological investigations were performed. The broadened site-specific database formed the basis for selecting the NL siting region for a repository with a section for spent fuel and high-level waste (SF/HLW) and a section for low- and intermediate-level waste (L/ILW), respectively, for which the general licence application has been prepared by the end of 2024. A provisional repository project for HLW and L/ILW at the Haberstal site was elaborated. The technical repository layout described in the engineering dossier (Nagra 2023a, Nagra 2023b) foresees the emplacement rooms for HLW and L/ILW to be placed in the Opalinus Clay formation at a depth of around 900 m.

The assessment of long-term safety of the repository requires a reliable representation of the excavation damage zone (EDZ) around the backfilled HLW and L/ILW emplacement rooms, which may form a preferential transport path for radionuclides after repository closure. The excavation and lining concept of the repository structures have been elaborated for the provisional repository project at the Haberstal site (Nagra 2023b), based on a comprehensive geomechanical database derived in the context of a dedicated site investigation program (Nagra 2024), (Nagra 2024b). In a previous performance assessment study (Nagra 2014), a comprehensive numerical workflow has been developed aimed at generating plausible (“representative”) EDZ fracture networks around the emplacement rooms of a generic HLW and L/ILW repository (Geomechanica 2013). These hypothetical EDZ fracture networks were used in a subsequent step to simulate fluid flow and radionuclide transport along the backfilled underground structures, drawing on a simplified but consistent representation of transmissive features of the EDZ (Alcolea et al. 2014).

The scope of this report is to update the previous workflow (Geomechanica 2018) by considering the latest site specific and layout specific databases, associated with the combined repository for HLW and L/ILW in the siting region NL. Probable variations of the EDZ fracture patterns are to be determined for HLW emplacement tunnels, L/ILW emplacement caverns and shaft. Simulated fracture patterns are to represent the expected shape and size of the EDZ as well as deviations thereof.

1.2 Previous related numerical modelling studies

The basic workflow for the evaluation of repository-induced effects by numerically simulating the creation of the EDZ during repository construction and its temporal evolution after backfilling of the underground structures is documented in (Geomechanica 2013). In particular, a finite-discrete element method (FDEM) was used to simulate the discrete fracture network which develops in response to the excavation process in the rock mass around the underground structures of a radioactive waste repository. The synthetic fracture networks were then fed into a hydraulic model (Alcolea et al. 2014), aimed at providing abstracted representations of the EDZ as an input for safety analyses.

A follow-up study of the above numerical investigations was carried out with results documented in (Geomechanica 2016a). The report complemented the results of previous laboratory-scale models by (i) assessing the sensitivity of FDEM models to a variety of input parameters and

boundary conditions and (ii) producing a series of characteristic plots suitable to perform continuum-based diagnostic analyses of geomechanical properties.

Another sensitivity study on the EDZ formation for L/ILW and SF/HLW emplacement tunnels at three candidate repository sites (Jura Ost (JO), Zürich Nordost (ZNO), and Nördlich Lägern (NL)) was performed in preparation of SGT – Stage 3. The sensitivity analysis included assessing the effect of varying the rock strength, in-situ stress field, and pore pressure on the formed EDZ. The results of the study are documented in (Geomechanica 2018) and (Alcolea 2018).

1.3 Scope and objectives

The purpose of this study was to simulate fracture patterns, representing the shape and extent of the EDZ of HLW, L/ILW, and shaft designs at 900 m depth using the latest available laboratory, field, and engineering design data. This study was summarized into the following tasks:

1. A review of relevant studies was performed to extract updated geomechanical data, geometric data, and modelling boundary conditions.
2. Using the calibration results from (Geomechanica 2013) and (Geomechanica 2016a) as a starting point, the Opalinus Clay strength properties were adjusted to reflect the new laboratory data by (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023). The updated input parameters were strictly a result of minor UCS calibration.
3. Perform 2D simulations (27 simulations in total) of three updated excavation designs for the geological waste repository at 900 m depth in the siting region NL of Northern Switzerland.
 - a) Sensitivity analysis of the EDZ around the designed HLW tunnel with varying in-situ stress, rock strength properties, and core softening routines. In total, nine simulations were performed.
 - b) Sensitivity analysis of the EDZ around the designed L/ILW cavern with varying in-situ stress, rock strength properties, and core softening routines. In total, nine simulations were performed.
 - c) Sensitivity analysis of the EDZ around the designed shaft with varying in-situ stress, rock strength properties, and core softening routines. In total, nine simulations were performed.

1.4 Report outline

Chapter 1 presents the background and motivations for the present study. The objectives are specified and the report organization is discussed.

Fundamentals of the FDEM modeling approach, as implemented in Geomechanica's Irazu software, are summarized in Chapter 2. This includes a brief synopsis of the software developments and basic formulations of the implemented deformation and fracturing algorithms.

Chapter 3 describes the methodology and the basic workflow applied for the simulation of the EDZ fracture networks.

Chapters 4, 5, and 6 display the simulation results obtained for the investigated repository components, namely the HLW tunnel, the L/ILW cavern, and the shaft. Results of the base case simulations and parameter variants are presented.

Chapter 7 summarizes the achievements and provides the main conclusions of the study.

2 Modelling software: Irazu

The hybrid finite/discrete element method (FDEM) is a numerical method which combines continuum mechanics principles with DEM algorithms to simulate multiple interacting deformable bodies (Munjiza 2004) (Munjiza, A., Owen, D. R. J., & Bicanic, N. 1995). The latest version of Geomechanica's FDEM software, Irazu V. 6.1, is used in this study (Geomechanica 2023). In FDEM, each solid is discretized as a mesh consisting of nodes and triangular elements. An explicit time integration scheme is applied to solve the equations of motion for the discretized system and to update the nodal coordinates at each simulation time step. In general, the governing equations can be expressed as:

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{f}_{\text{int}}(\mathbf{x}) - \mathbf{f}_{\text{ext}}(\mathbf{x}) - \mathbf{f}_c(\mathbf{x}) - \mathbf{f}_h(\mathbf{x}) = 0, \quad (1)$$

where $\mathbf{M}$ and $\mathbf{C}$ are the system mass and damping diagonal matrices; $\ddot{\mathbf{x}}$, $\dot{\mathbf{x}}$, and $\mathbf{x}$ are the vectors of nodal accelerations, velocities, and displacements; $\mathbf{f}_{\text{int}}$ is the vector of internal resisting forces, $\mathbf{f}_{\text{ext}}$ is the vector of applied external loads, $\mathbf{f}_c$ is the vector of contact forces, $\mathbf{f}_h$ is the vector of fluid-pressure forces. Contact forces, $\mathbf{f}_c$, are calculated either between contacting discrete bodies or along internal discontinuities (i.e., pre-existing or newly created fractures) (Section 2.1). Internal resisting forces, $\mathbf{f}_{\text{int}}$, include the contribution from the elastic forces, $\mathbf{f}_e$, and the crack element bonding forces, $\mathbf{f}_b$, which are used to simulate material elastic deformation and progressive failure, as further explained in Sections 2.2 and 2.3.

Numerical damping is introduced in the governing equation to account for energy dissipation due to non-linear material behaviour or to model quasi-static phenomena by dynamic relaxation (Munjiza 2004)). The matrix $\mathbf{C}$ is equal to:

$$\mathbf{C} = k_s \mathbf{I}, \quad (2)$$

where k_s is a constant damping coefficient and $\mathbf{I}$ is the identity matrix.

2.1 Contact detection and interaction of triangular finite elements

A FDEM simulation can comprise a very large number of potentially interacting distinct elements. To correctly capture this behaviour, contacting couples (i.e., pairs of contacting discrete elements) must first be detected. Subsequently, the interaction forces, $\mathbf{f}_c$, resulting from such contacts can be defined. Contact interaction forces are calculated between all pairs of elements that overlap in space. Two types of forces are applied to the elements of each contacting pair: repulsive forces and frictional forces. The repulsive forces between the elements of each contacting pair are calculated using a penalty function method (Munjiza, A., & Andrews, K. R. F. (2000)). The contacting finite elements penetrate into each other, thus generating distributed reaction forces that depend on the shape and size of the resulting overlapping area. The stiffness of the contact is represented by the normal penalty coefficient, p_n . As the value of p_n tends to infinity, a body impenetrability condition is approached. The frictional forces between contacting couples are calculated using a Coulomb-type friction law. These frictional forces are used to simulate the shear strength of intact material and of pre-existing and newly created fractures (Mahabadi et al., 2012). These frictional forces are applied to intact material, pre-existing discontinuities and newly-created fractures. The frictional resistance is mobilized over a finite amount of relative sliding between the interacting element edges. This finite displacement depends upon a tangential

stiffness parameter, namely the tangential penalty coefficient, p_t . The frictional resistance is equal to:

$$f_s = \sigma_n \tan \varphi_f, \quad (3)$$

where σ_n is the normal stress acting across the element edges and φ_f is the fracture friction angle.

2.2 Deformation of triangular finite elements

Since the material strain is expected to be localized in the crack elements (see Section 2.3), the bulk material is treated as linear-elastic using constant-strain triangular finite elements. To capture the stiffness anisotropy of the Opalinus Clay, a stress-strain constitutive law for a linearly elastic, transversely isotropic medium is typically adopted. With this model, the elastic deformation is fully characterized by five independent elastic constants (Jaeger, J. C., & Cook, N. G. W. 1976)) (Fig. 2-1):

- E_t and E_l are the Young's moduli in the direction parallel and perpendicular to the plane of transverse isotropy.
- ν_{tl} and ν_{lt} are the Poisson's ratios that characterize the transverse strain in the plane of isotropy due to a stress applied in the direction parallel and perpendicular to it.
- G is the shear modulus for planes normal to the plane of transverse isotropy.

In general, the elastic response of a FDEM model depends not only on the constitutive relationship of the triangular elements but also on the properties of the crack elements. Since the elastic deformation before the onset of fracturing takes place in the bulk material, no deformation should in theory occur in the crack elements before the intrinsic strength is exceeded. However, a finite stiffness is required for the crack elements by the time-explicit formulation of FDEM. Such an artificial compliance is represented by the normal, tangential and fracture penalty values, p_n , p_t , and p_f , for compressive, shear, and tensile loading conditions. For practical purposes, the contribution of crack elements to the overall model compliance can be largely limited by adopting very high penalty values (Mahabadi et al. 2012, Tatone & Grasselli 2015). Consequently, no anisotropy in deformation is introduced in the penalty formulation at the crack element level and the elastic response is effectively controlled by the choice of the stress-strain relationship of the continuum triangular elements.

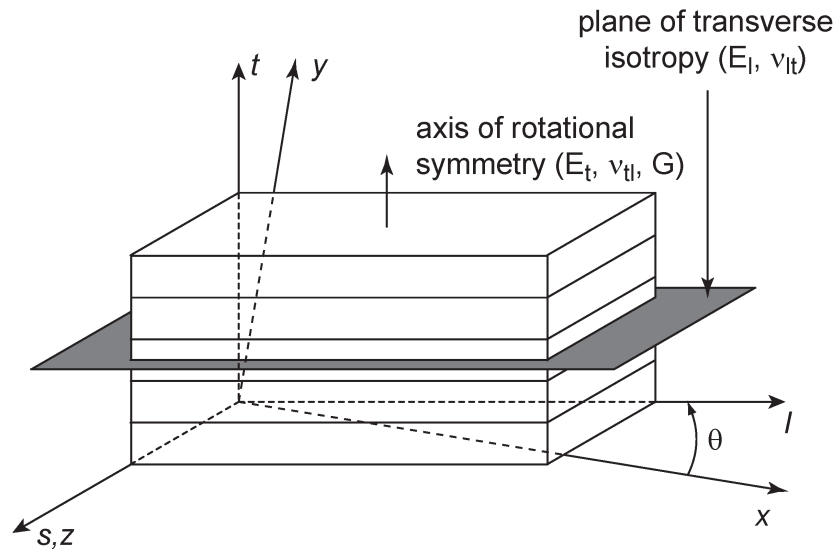


Fig. 2-1: Transversely isotropic material with associated elastic constants

2.3 Deformation and fracturing of crack elements

The progressive failure of rock material is modelled using a cohesive-zone approach, a technique first introduced in the context of the elasto-plastic fracturing of ductile metals (Barenblatt 1962, Dugdale 1960). This approach aims at capturing the non-linear interdependence between stresses and strain that characterizes the zone ahead of a macro-crack tip known as the Fracture Process Zone (FPZ). As depicted in Fig. 2-2, the FPZ in brittle rocks manifests itself in the form of micro-cracking and interlocking related to the presence of micro-scale inhomogeneities (e.g., mineral grains and pre-existing defects or voids) (Labuz et al. 1987)). When using cohesive-zone models, the failure of the material progresses based solely on the strength degradation of dedicated interface elements (referred herein to as crack elements) and therefore emerges as a natural outcome of the deformation process without employing any additional macroscopic failure criterion.

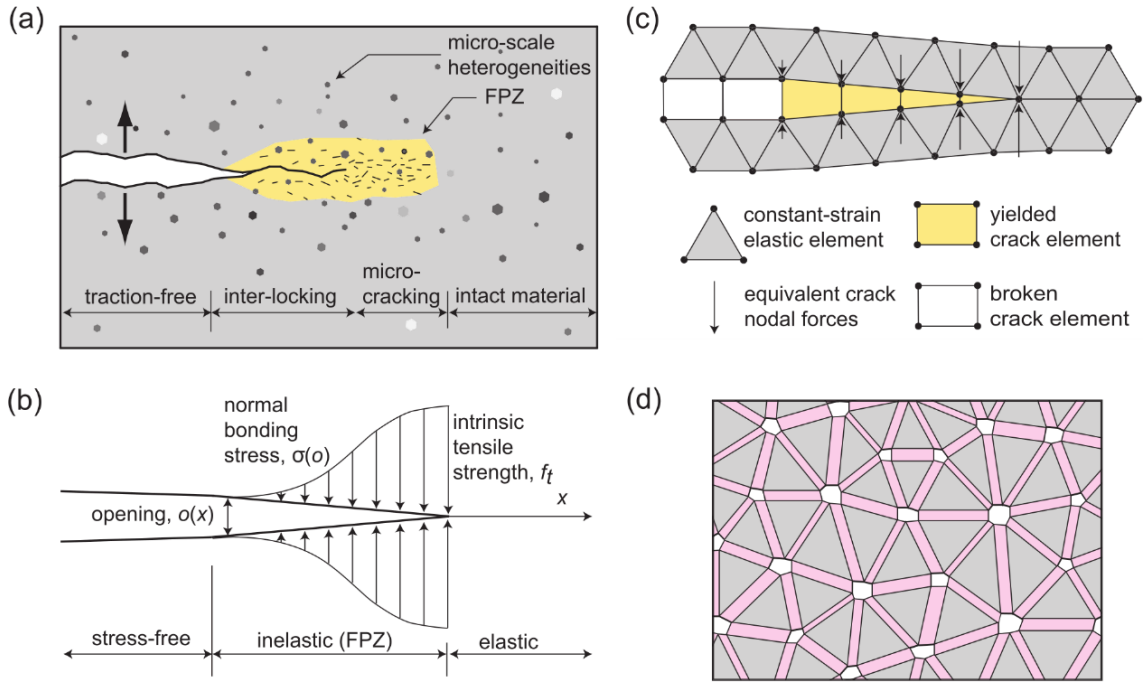


Fig. 2-2: Cohesive-zone approach for material failure modelling in FDEM

(a) Conceptual model of a tensile crack in a heterogeneous rock material (after (Labuz et al. 1985), modified). (b) Theoretical FPZ model (Hillerborg et al., 1976). (c) Numerical representation of the theoretical FPZ model in Irazu (after (Lisjak et al. 2013)). (d) Exaggerated view of the four-noded cohesive crack elements located at adjoining edges of triangular finite elements (after (Lisjak et al. 2013)).

In the Irazu FDEM code, the bonding stresses transferred by the material are decreasing functions of the displacement discontinuity across the crack elements according to the cohesive laws illustrated in Fig. 2-3. These constitutive relationships represent a modified version of the crack model response proposed by (Munjiza, A., Andrews, K. R. F., & White, J. K. 1999). Mode I (i.e., opening) fracturing Fig. 2-3a) is simulated by a cohesive model based on the FPZ model originally developed for concrete by Hillerborg under the name of fictitious crack model (Hillerborg et al. 1976)). A fracture is assumed to initiate when the crack tip opening, o , reaches a critical value, o_p , related to the intrinsic tensile strength of the material, f_t . When the crack opens, the normal bonding stress, σ , is not assumed to fall to zero instantaneously, but to decrease with increasing crack opening until a residual opening value, o_r , is reached and a stress-free surface is created (i.e., $\sigma = 0$). Mode II (i.e., shear) fracturing (Fig. 2-3b) is simulated by a slip-weakening model conceptually similar to that of (Ida 1972). For shear loading conditions, a tangential bonding stress, τ , exists between the two fracture walls, which is a function of the amount of slip, s , and the normal stress on the fracture, σ_n . The critical slip, s_p , corresponds to the intrinsic shear strength of the rock, f_s , defined as:

$$f_s = c + \sigma_n \tan \varphi_i, \quad (4)$$

where c is the internal cohesion, φ_i is the material internal friction angle, and σ_n is the normal stress acting across the crack element. Upon undergoing the critical slip, s_p , the tangential bonding stress is gradually reduced to a residual value, f_r , which corresponds to purely frictional resistance:

$$f_r = \sigma_n \tan \varphi_f, \quad (5)$$

where φ_f is the fracture friction angle and σ_n is the normal stress acting across the fracture surfaces. In the current crack element implementation, the unloading path in the softening branch coincides with the loading path. Therefore, the model is strictly only valid for monotonic loading conditions.

For mixed mode fracturing, the rupture of a crack element is defined by the following coupling criterion between crack opening and slip (Fig. 2-3c):

$$\left(\frac{o - o_p}{o_r - o_p} \right)^2 + \left(\frac{s - s_p}{s_r - s_p} \right)^2 \geq 1. \quad (6)$$

The mode of fracture, m , associated with a broken crack element can be approximated as:

$$m = 1 + \frac{s - s_p}{s_r - s_p}. \quad (7)$$

The external energy required to fully break a unit surface area of cohesive crack corresponds to the input specific fracture energy, G_c . G_c is defined in terms of the material properties G_{Ic} and G_{IIc} which correspond to the strain energy release rates for Mode I and Mode II fracturing. They are equal to the area under the curves in Fig. 2-3a,b given by:

$$G_{Ic} = \int_{o_p}^{o_r} \sigma(o) do, \quad (8)$$

$$G_{IIc} = \int_{s_p}^{s_r} \tau(s) ds. \quad (9)$$

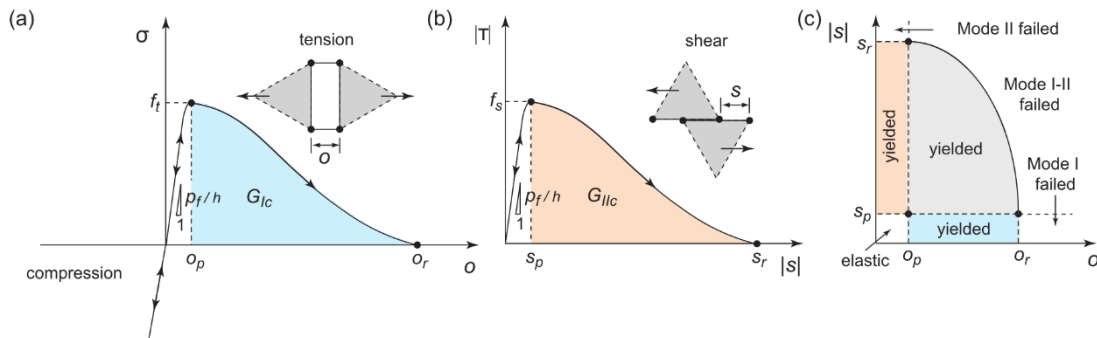


Fig. 2-3: Fracture model implemented in FDEM

(a) cohesive model for Mode I, (b) slip-weakening model for Mode II, and (c) criterion for mixed-Mode fracture initiation. Note that the shape and scale of the post-peak curves drawn in (a) and (b) are approximated for illustration purposes only: the reader is referred to the text for an accurate mathematical description.

Following an approach similar to that pioneered by (Xu & Needleman 1996), the crack elements in FDEM are interspersed throughout the material (i.e., across the edges of all triangular element pairs) from the very beginning of the simulation. Thus, cracks are allowed to nucleate and grow without any additional assumption or criterion other than the crack element constitutive response. Upon breakage of the cohesive surface, the crack element is removed from the simulation and therefore the model transition from a continuum to discontinuum is locally completed. The newly created discontinuity is treated by the contact algorithm through the contact forces, $\mathbf{f}_c$, briefly described in the previous section. As the simulation progresses, finite displacements and rotations of discrete bodies are allowed and new contacts are automatically recognized.

In this study, the simulation of strength anisotropy followed the approach first developed by (Lisjak et al. 2014) to model the mechanical response of Opalinus Clay and originally implemented in the two-dimensional Y-Geo code (Mahabadi et al. 2012)). Specifically, anisotropy of strength is introduced at the crack element level by imposing that the cohesive strength of each crack element is a function of the relative orientation, γ , between the crack element itself and the bedding orientation (Fig. 2-4a). The cohesive strength parameters and the fracture energy release rates are assumed to vary linearly between a minimum value for $\gamma = 0^\circ$ (i.e., $f_{t,min}$, c_{min} , $G_{Ic,min}$, $G_{IIc,min}$) to a maximum value for $\gamma = 90^\circ$ (i.e., $f_{t,max}$, c_{max} , $G_{Ic,max}$, $G_{IIc,max}$). Furthermore, the mesh topology combines a random triangulation for the intra-layer material (i.e., matrix) together with crack elements preferably aligned along the bedding planes (Fig. 2-4b).

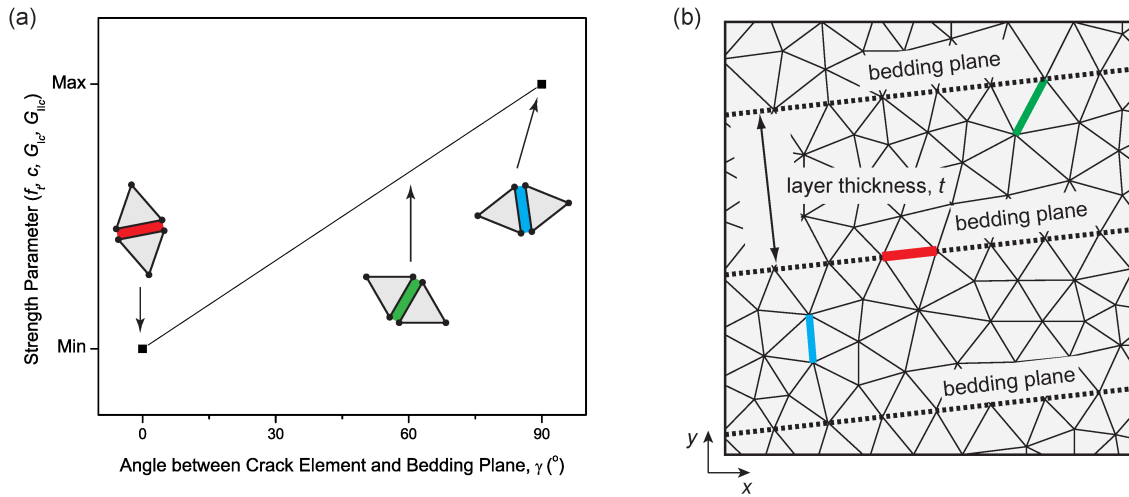


Fig. 2-4: FDEM modelling of strength anisotropy

(a) Linear variation of cohesive strength parameters with the angle, γ , between crack element and bedding. (b) Example of mesh combining a Delaunay triangulation for the intra-layer material with edges preferentially aligned along the bedding plane direction.

3 Modelling methodology

3.1 Geometries and mesh generation

Three main model geometries were considered for the EDZ simulations: (i) a 3.7 m diameter circular opening for the HLW emplacement tunnel (Fig. 3-1), (ii) a 12.4 m $\times$ 13.7 m horseshoe-shaped opening for the L/ILW emplacement cavern (Fig. 3-2), and (iii) an 8.5 m diameter circular opening for the shaft (Fig. 3-3). The HLW, L/ILW, and shaft excavations were placed at the centre of a 100 m $\times$ 100 m, 200 m $\times$ 200 m, and 200 m $\times$ 200 m exterior domain. The cross-section geometries of the excavations were provided by Nagra in the latest design revision to date. The cross-sections of the HLW and L/ILW excavations were assumed to be perpendicular to the strike of flat-lying bedding planes (i.e., $\psi = 0^\circ$). Thus, the rock mass behaviour was captured using an anisotropic strength and stiffness model. The cross-section of the shaft was assumed to be parallel to the strike of flat-lying bedding planes (i.e., in the plane of isotropy), and thus the rock mass behaviour was captured using an isotropic strength and stiffness (plane strain) model. The shaft models utilized elastic properties in the direction parallel to bedding (i.e., E_l , ν_{lt}) and the greater strength values (i.e., $f_{t,max}$, c_{max} , $G_{lc,max}$, $G_{llc,max}$).

To maximize the model resolution in the EDZ while keeping the run times within practical limits, a mesh refinement zone of 13 m $\times$ 13 m, 50 m $\times$ 50 m, and 50 m $\times$ 50 m was adopted around the excavation boundary, with average element size, h , of 0.04 m, 0.15 m, and 0.15 m for the HLW, L/ILW, and shaft models. From the refinement zone, the element size was graded towards the external boundaries, where an element size equal to 5 m, 10 m, and 10 m was used for the HLW, L/ILW, and shaft models. The finite element mesh was also graded internally from the inner surface of the lining ($h = 0.04$ m, $h = 0.15$ m, and $h = 0.15$ m) towards the center of the excavations ($h = 0.2$ m, $h = 0.7$ m, and $h = 0.4$ m).

Based on the study conducted in (Geomechanica 2016b), the HLW and L/ILW models had a mesh bias implemented to replicate the depositional structure of the Opalinus Clay. For the HLW and L/ILW models, a non-persistent, horizontal, discrete fracture network (DFN) was embedded in the mesh with the following properties in Tab. 3-1.

Tab. 3-1: DFN parameters for embedded, non-persistent joints in the refinement region around the excavation

Parameter	HLW	L/ILW
Segment length, l (m)	0.35	1.20
Bridge length, b (m)	0.28	0.96
Layering thickness, t (m)	0.12	0.40
Orientation, ψ ($^\circ$)	0	0

The total number of triangular finite elements employed was approximately equal to 286,000, 315,000, and 320,000 for the HLW, L/ILW, and shaft models. Equations of motion for the discretized systems were integrated with a time step of 1.7×10^{-7} , 6.2×10^{-7} s, and 5.4×10^{-7} s for the HLW, L/ILW, and shaft models; these values were the largest time step sizes that ensured numerical stability for the explicit solver of the code (based on the adopted elastic parameters and finite element size).

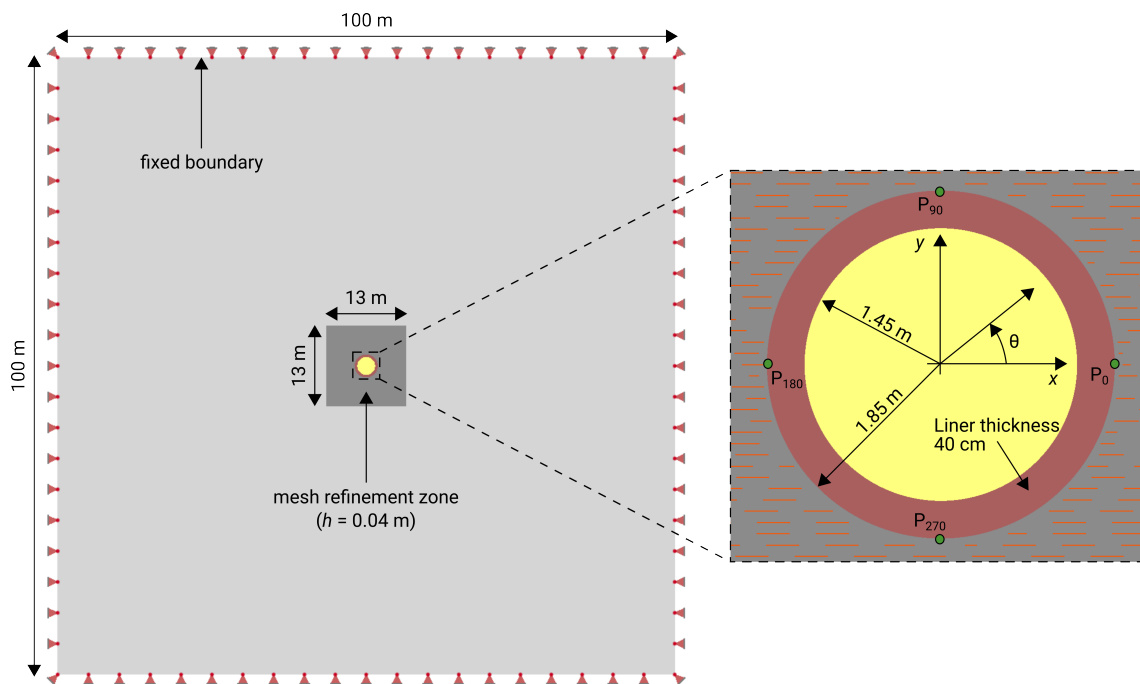


Fig. 3-1: Geometry and applied boundary conditions of the HLW tunnel

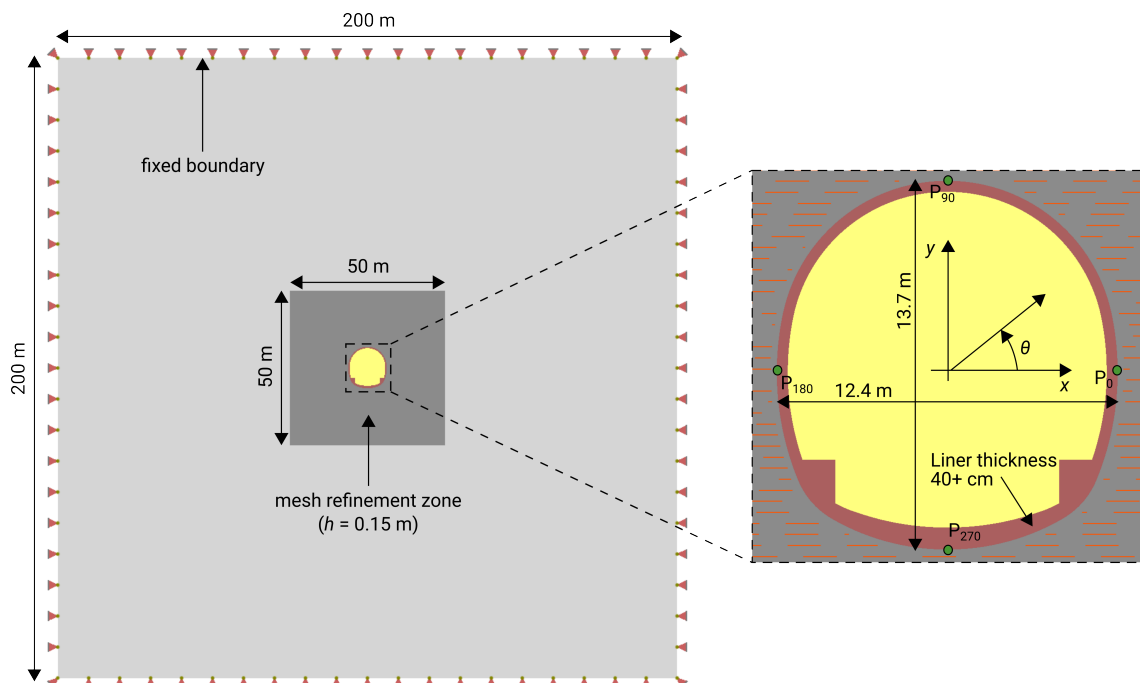


Fig. 3-2: Geometry and applied boundary conditions of the L/ILW cavern

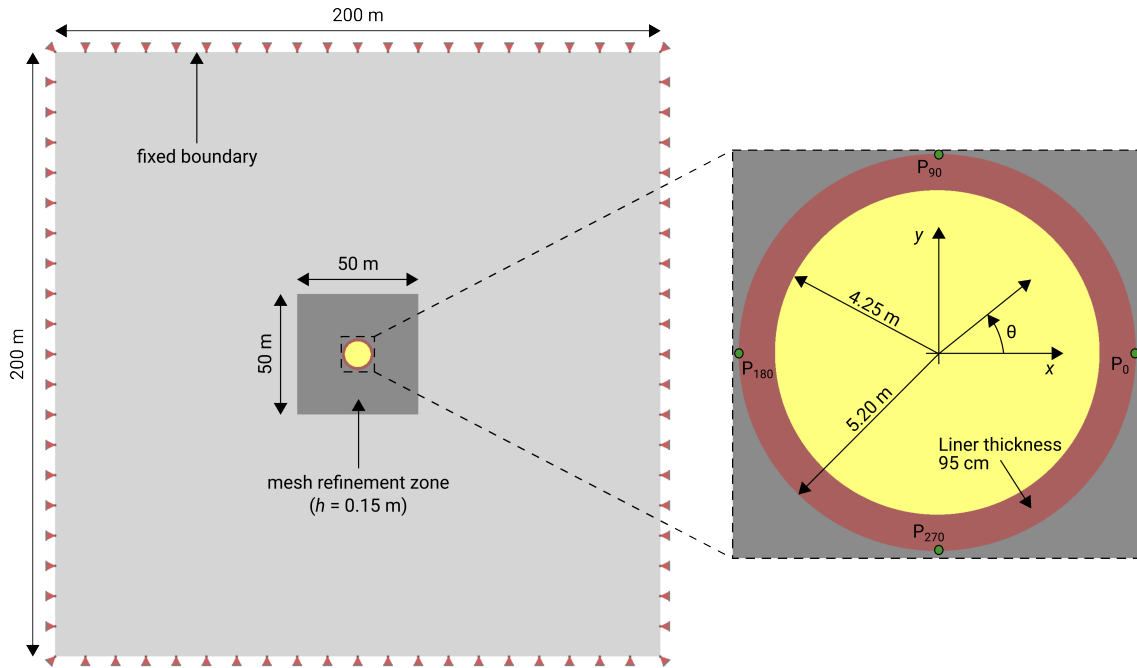


Fig. 3-3: Geometry and applied boundary conditions of the shaft

3.2 Rock input parameters and calibration

The rock input parameters used in this study are summarized in Tab. 3-2. The elastic properties of the rock were directly adopted from (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023). Minor adjustment of the minimum and maximum cohesion from (Geomechanica 2016b) was necessary to match new laboratory test data for Opalinus Clay.

From the triaxial laboratory data shown in (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023) the general trend is that the rock, in compression, is stronger when loaded in the orientation parallel to the bedding planes (P-sample) than when loaded in the orientation perpendicular to the bedding plane (S-sample) (Fig. 3-4). This exhibited behaviour is the opposite of the underlying assumption in Irazu's anisotropic strength formulation (Fig. 2-4), that is, the rock is weaker when loaded parallel to bedding than perpendicular to bedding, and therefore, it is not possible to reproduce the laboratory behaviour using this model. As a compromise, it was decided that the property sets will have more isotropic compressive strengths, but will have an upper bound that matches the extrapolated uniaxial compressive strength (UCS) of the P-sample, and a lower bound that matches the extrapolated UCS of the S-sample.

The calibration was performed through a trial-and-error process by adjusting the minimum and maximum cohesion of the rock and simulating the emergent UCS of three models: P-sample, S-sample, and Z-sample (loading 30° to bedding). The UCS models use the same geometrical parameters as (Geomechanica 2016b) and are summarized in Tab. 3-3. The model is purely mechanical and the pore pressure in the rock was ignored. The mesh is preconditioned with a non-persistent joint network to mimic the bedding plane of the rock (12). Loading is applied to the rock using two rigid platens with a constant velocity of 0.05 m/s. Although this loading rate is significantly greater than that typically used in actual experiments, it was verified by (Tatone & Grasselli 2015) that emergent UCS values converge at effective loading rates less than 0.25 m/s.

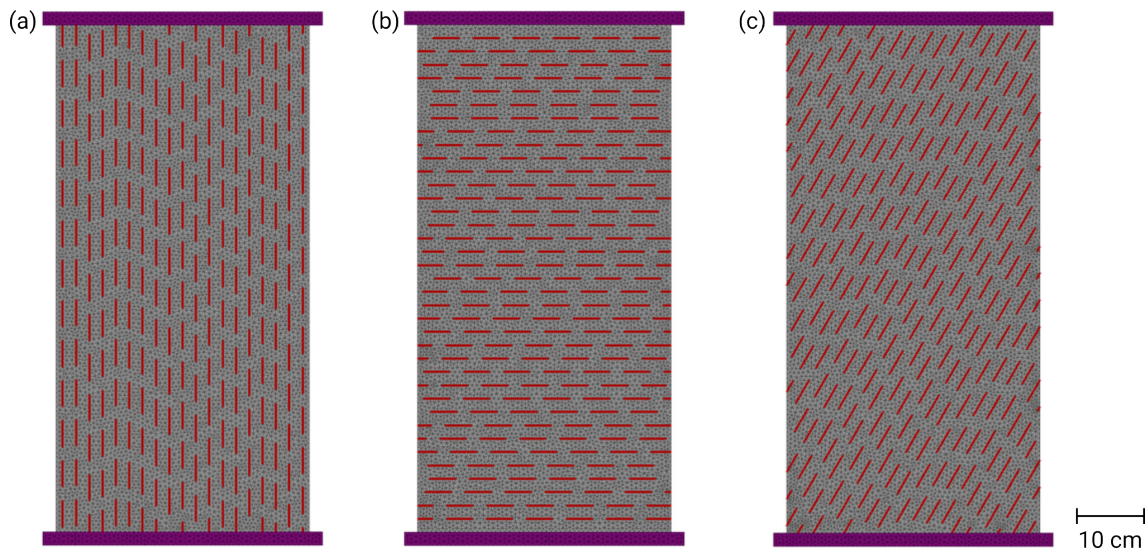


Fig. 3-4: The (a) P-sample, (b) S-sample, and (c) Z-sample (30° bedding) mesh, with the geometric parameters summarized in Tab. 3-3, used for input parameter calibration

A comparison between emergent UCS from previous studies, in addition to the newly calibrated lower and upper bound strength parameters are shown in Tab. 3-4. In addition, the simulated stress-strain plots and emergent fracture patterns are shown in Fig. 3-5. Overall, the simulated UCS of the P- and S-sample are more isotropic using the current calibration as the difference between the minimum and maximum cohesion is 0.5 MPa to 1.5 MPa, which is much less than the calibration from (Geomechanica 2016b) which was 16.25 MPa. In the upper bound strength scenario, the S- and Z-sample UCS are much greater than the laboratory data by (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023), but there is good match for the P-sample UCS. In the lower bound strength scenario, there is good match between the S- and Z-sample UCS, but the simulated P-sample UCS is roughly half of the extrapolated laboratory data. In terms of failure, P-sample and S-sample UCS simulations exhibit inter-bedding shearing, and Z-samples exhibited shear failure through the bedding. The new sets of calibrated input parameters were deemed satisfactory to simulate the upper and lower limit of EDZ shape and extent in accordance with laboratory data.

Tab. 3-2: Rock input parameters for Opalinus Clay

Parameter	Lower bound	Upper bound
<i>Triangular finite elements</i>		
Bulk density ¹ , ρ (kg·m ⁻³)	2,430	2,430
Young's modulus parallel to bedding ² , E_l (GPa)	9.89	9.89
Young's modulus perpendicular to bedding ² , E_t (GPa)	5.78	5.78
Poisson's ratio parallel to bedding ² , ν_{lt} (-)	0.27	0.27
Poisson's ratio perpendicular to bedding ² , ν_{tl} (-)	0.28	0.28
Shear modulus ² , G (GPa)	2.5	2.5
Viscous damping factor ³ , (-)	1	1
<i>Crack elements</i>		
Tensile strength parallel to bedding ¹ , $f_{t,max}$ (MPa)	2	2
Tensile strength perpendicular to bedding ¹ , $f_{t,min}$ (MPa)	0.32	0.32
Cohesion parallel to bedding ⁴ , c_{min} (MPa)	3.5	7.5
Cohesion perpendicular to bedding ⁴ , c_{max} (MPa)	4.0	9.0
Mode I fracture energy parallel to bedding ¹ , $G_{Ic,max}$ (Jm ⁻²)	5	5
Mode I fracture energy perpendicular to bedding ¹ , $G_{Ic,min}$ (Jm ⁻²)	14	14
Mode II fracture energy parallel to bedding ¹ , $G_{IIc,min}$ (Jm ⁻²)	20	80
Mode II fracture energy perpendicular to bedding ¹ , $G_{IIc,max}$ (Jm ⁻²)	23	96
Friction angle of intact material ¹ , ϕ_i (°)	24	24
Friction angle of fractures ¹ , ϕ_f (°)	24	24
Normal contact penalty ⁵ , p_n (GPa·m)	989	989
Tangential contact penalty ⁵ , p_t (GPa/m)	98,900	98,900
Fracture penalty ⁵ , p_f (GPa)	989	989

¹ Value from (Geomechanica 2016a)² Value from (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023)³ Critical damping coefficient based on finite element size and elastic properties⁴ Valued obtained through iterative, trial-and-error procedure (see (Geomechanica 2013),(Geomechanica 2016a) for further details)⁵ Values chosen to minimize the influence of the crack element compliance on the (tangential and normal) stiffness of the model based on the latest Irazu 2D formulation

Tab. 3-3: Geometrical parameters of UCS calibration models from (Geomechanica 2013), (Geomechanica 2016a)

Property	Value
Diameter (mm)	38
Length (mm)	76
Nominal element size, h (mm)	0.7
Segment length, l (mm)	4
Bridge length, b (mm)	2
Layering thickness, t (mm)	2

Tab. 3-4: Results of UCS data from previous studies and current calibration

Report	UCS P-sample [MPa]	UCS S-sample [MPa]	UCS Z-sample [MPa]
Extrapolated lab data – Madaschi et al. (2023)	27.7	14.5	10.7 ^{1,2}
Simulation result – NAB 16-01 (Geomechanica Inc., 2016)	24.7	36.4	5.6 ³
Simulation result – upper bound calibration in current study	26.7	28.5	19.0 ¹
Simulation result – lower bound calibration in current study	12.7	13.4	9.3 ¹

¹ Loading 30° to bedding

² Outlier Z-sample data removed in extrapolation

³ Loading 45° to bedding

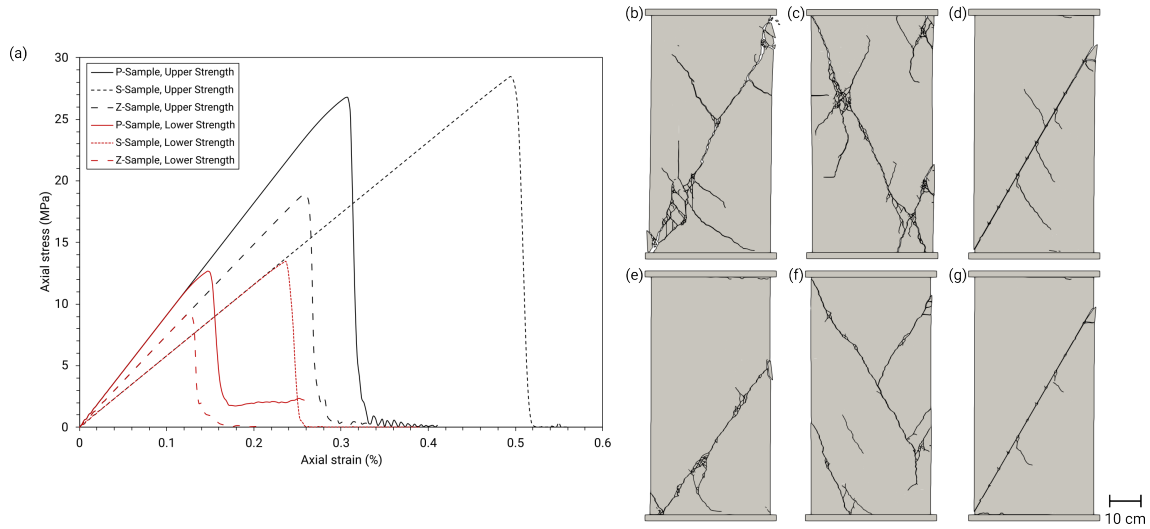


Fig. 3-5: The stress-strain plots of the UCS calibration simulations.

(a) The post-peak fracture patterns of the upper strength (b) P-sample, (c) S-sample, and (d) Z-sample and lower strength (e) P-sample, (f) S-sample, and (g) Z-sample.

3.3 In-situ stress and boundary conditions

The in-situ stress fields used for this study are based on a depth of 900 m where the vertical stress, $\sigma_v = 22.5$ MPa. Three cases of the horizontal in-situ stress are considered: (i) $\sigma_h/\sigma_v = 1$ (i.e., isotropic; $\sigma_h = 22.5$ MPa), (ii) $\sigma_h/\sigma_v = 0.77$ (i.e., $\sigma_h = 17.5$ MPa), and (iii) $\sigma_h/\sigma_v = 1.3$ (i.e., $\sigma_h = 29.25$ MPa). For shaft simulations, one of the horizontal stresses was equal to 22.5 MPa and the same in-situ stress ratios were used to determine the magnitude of the other in-plane in-situ stress.

Since the adopted crack elements do not account for the influence of an out-of-plane stress during the simulation of fracture nucleation and growth, only the in-plane stresses were effectively used in the analysis. Also, gravity-induced stress gradients were neglected in all models. The initialization of each model with the prescribed in-situ stress field was accomplished by a preliminary elastic run (i.e., without crack elements) which aimed at obtaining the revised nodal coordinates of the non-excavated model subjected to the prescribed far-field stresses. These revised nodal coordinates were subsequently used as the current nodal coordinates (i.e., deformed mesh) of the actual FDEM simulation run in which crack elements were inserted into the model to capture material failure. By changing the far-field boundaries to be fixed in the horizontal and vertical directions, the first order in-situ stress conditions were maintained while allowing the excavation sequence to be modelled.

3.4 Incorporation of effective stresses

For each model, the pore pressure field was assumed homogeneous throughout the domain with pressure value computed at the excavation depth based on hydrostatic conditions with groundwater table located at the ground surface. The pore pressure gradient is 0.01 MPa/m, equating to 9 MPa of pore pressure at 900 m depth. To obtain effective stress conditions throughout the model, the fluid pressure is applied as a surface pressure to the two adjoining edges of all finite

element pairs (i.e., inside each crack element, see Fig. 3-6). The pore pressure was applied to each crack element regardless of their state (i.e., intact/elastic, softened, or failed). From a fracture analysis perspective, a pore pressure, P_w , reduces the effective normal stress:

$$\sigma'_n = \sigma_n - P_w, \quad (10)$$

acting along each crack element and consequently the mobilized frictional resistance:

$$f_s = \sigma'_n \tan \phi_f. \quad (11)$$

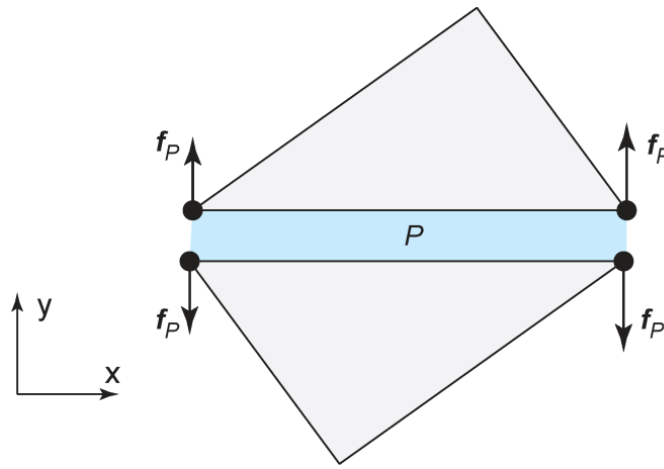


Fig. 3-6: Incorporation of effective stresses by application of fluid-pressure-induced forces as surface pressure to the edges of adjoining finite element pairs

Initialization of the model with the pore pressure field was performed right after the in-situ stress initialization stage. Upon initiation of the excavation sequence, the fluid pressure along the excavation boundary is set to zero. The pore pressure in the rock is allowed to change in response to the mechanical deformation of the crack elements (i.e., one-way hydro-mechanical coupling). As the aperture of the crack element increases (or decreases), the cavity volume also increases (or decreases), and hence, there is a reduction (or increase) in pore pressure. The rate at which the pore pressure changes is controlled by the fluid bulk modulus (K_f). While marching in time, the pore pressure is calculated as:

$$P_w^t = P_w^{t-1} + K_f \frac{V_c^t - V_c^{t-1}}{V_c^t + V_c^{t-1}/2}, \quad (12)$$

where P_w^t and P_w^{t-1} is the pore pressure at time t and $t-1$, and V_c^t and V_c^{t-1} is the cavity volume at time t and $t-1$. The K_f used in all models is 0.05 GPa and is much less than that of water (2.2 GPa). Numerically, this is necessary to minimize dynamic effects in the model attributed to a sudden loss of pore pressure in response to fracturing. Pore pressure variations related to mass transport (i.e., fluid flow) and other coupled hydro-mechanical processes such as poroelasticity were not accounted for.

An alternative modelling approach for applying pore pressure to the model was to use a constant pore pressure boundary condition. This methodology was abandoned as it yielded unrealistic dynamic effects on the liner material upon excavation and this is further described in App. A.1.

3.5 Modelling procedure for excavation advance

To capture the supporting effect of the tunnel face in the two-dimensional FDEM model, a core replacement technique was adopted for this study based on the approach first developed by (Curran, J. H., Hammah, R. E., & Yacoub, T. 2003). With this technique, the three-dimensional face effect, which causes a gradual reduction of radial resistance around the excavation boundary, is captured by a fictitious, softening elastic material in the tunnel core. That is, the deformation modulus of the excavated material is progressively reduced from the original rock mass values, corresponding to an undeformed section far ahead of the face, to a value that results in the wall displacements at the time of support installation. The general procedure for modelling excavation advance and support installation in FDEM consisted of two main steps, graphically summarized in Fig. 3-7 (see also App. B).

Using the core replacement technique, the modulus reduction necessary to obtain the prescribed amount of tunnel wall displacement needs to be simulated. To this end, a FDEM simulation was run whereby the elastic parameters of the fictitious core material (i.e., E_t , E_r , G) were reduced by a factor α_s (core softening ratio) decreasing over time (from 1 to 0), and the wall displacements recorded. The result was the plot of the tunnel wall displacements, δ , as function of α_s . The excavation boundary was allowed to relax up to the point where the prescribed wall displacements were obtained and subsequently the liner was activated. The maximum allowed total wall closures, as provided by (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023), were 6.1 cm, 20.5 cm, and 17.2 cm for the HLW tunnel, L/ILW cavern, and shaft excavations, which were approximately 3.3% of the excavation radius or half the horizontal span of the L/ILW cavern (Fig. 3-8). This corresponds with the short-term sidewall convergence at zero support pressure in Fig. 3-8, additionally the short-term crown convergence (1.2%), which is 2.2 cm, 8.5 cm, and 6.2 cm for the HLW, L/ILW, and shaft excavations, was also considered in all simulation plots for comparison.

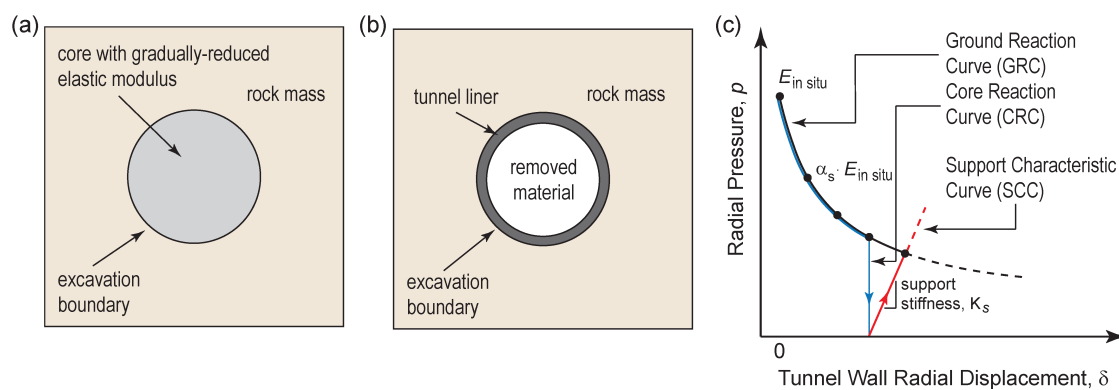


Fig. 3-7: Modelling procedure for excavation advance and support installation using the core replacement technique (see also App. B)

(a) Replacement of core with reduced elastic modulus material. (b) Installation of tunnel lining. (c) Qualitative example of ground reaction curve simulated by modulus reduction and support characteristic curve for a linear elastic lining.

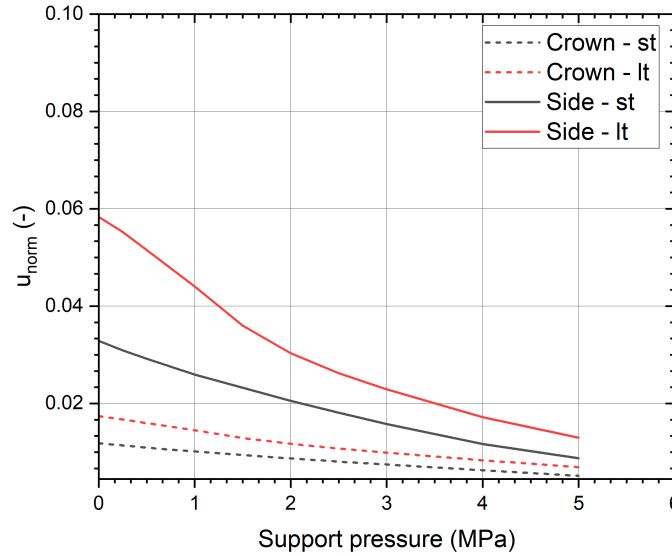


Fig. 3-8: Normalized short-term (st) and long-term (lt) ground reaction curves provided by Nagra used to determine excavation convergence before liner installation (from (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023))

Two core softening schemes for the HLW tunnel and L/ILW cavern were adopted in this study:

- 1. isotropic: E_t , E_r , G were reduced by the same α_s ; and
- 2. anisotropic: E_l and G were reduced by α_{sl} and E_t was reduced by α_{st} .

For six simulation cases, the isotropic core softening scheme was used and the original stiffness properties of the rock mass were linearly reduced by α_s . The simulation results based on the isotropic core softening scheme showed that the crown converges more than the sidewalls of the excavation which was the opposite of the provided GRC by Nagra (Fig. 3-8). Hence, an anisotropic core softening scheme was also adopted for three simulation cases to match both the magnitude and location of convergence as per the GRC provided by (Madaschi, A., Leuthold, J., Cantieni, L., & Laloui, L. 2023).

It should be noted that the use of an alternative excavation procedure, called the force relaxation method, was also investigated. Ultimately, this method was not used in this study for several reasons as further discussed in App. A.3.

3.6 Modelling procedure for support installation

Lining installation along the excavation perimeter was modelled using the constant-strain, triangular elements implemented in Irazu. Since the constitutive response of the support is at present limited to isotropic, linear elastic conditions (i.e., damage/fracturing is not allowed), the mechanical behaviour of the concrete liner was fully characterized by its Young's modulus, $E_c = 40$ GPa and Poisson's ratio, $\nu_c = 0.20$ for HLW models, and $E_c = 30$ GPa and $\nu_c = 0.20$ for L/ILW and shaft models.

The support installation consisted of specifying the liner thickness, t_c , and the installation time from the appropriate core softening ratio, α_s . Since the delayed installation of the support was accomplished by varying the elastic properties of the liner from those of the rock mass to those of the concrete (Fig. 3-7b), the deformation in the liner had to be zeroed to avoid an artificial build-up of stress in response to an instantaneous increase of material stiffness in a pre-stressed

medium. This was accomplished by a dedicated routine which sets the nodal coordinates of the deformed configuration (i.e., at the installation time) as new initial (i.e., undeformed) coordinates for the triangular mesh of the support.

3.7 Monitoring points of excavation boundary displacements

Temporal evolution of excavation wall convergence during the core modulus reduction sequence and subsequent lining activation was monitored in detail using four output history points designated as P_0 , P_{180} , P_{90} and P_{270} (Fig. 3-9). For HLW tunnel and L/ILW cavern models, $P_{0/180}$ were the sidewalls of the excavation, P_{90} was the crown, and P_{270} was the floor. Maximum allowed closure values were accounted for by examining the displacement components in the direction perpendicular to the excavation walls and allowing the excavation core material to relax up until the point in which one (or more) displacement components (in absolute terms) reached an approximate value of about 6.1 cm, 20.5 cm, and 17.2 cm, for HLW, L/ILW, and shaft models. For the HLW and shaft models, due to the circular shape of the excavation, the radial displacement, δ_r , is monitored, while for the L/ILW model, the x-displacement, δ_x , is monitored for P_0 and P_{180} , while the y-displacement, δ_y , is monitored for P_{90} and P_{270} .

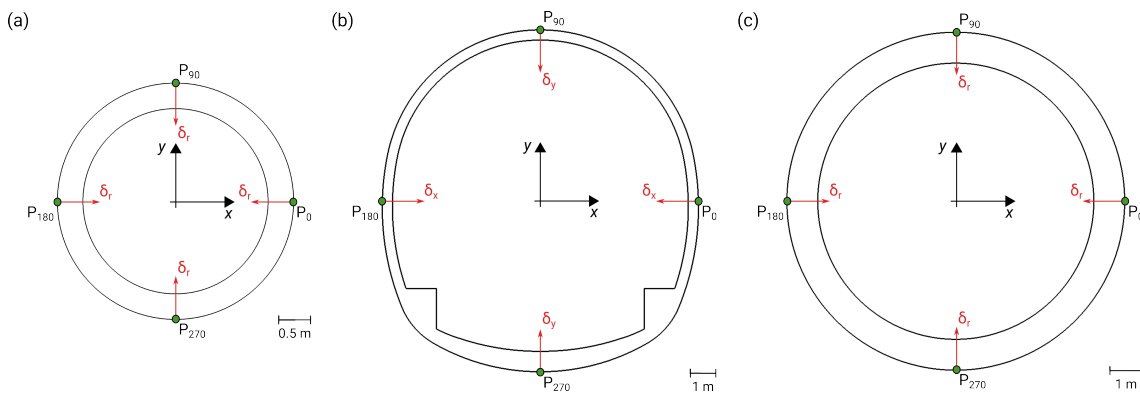


Fig. 3-9: Output history points used to monitor excavation wall displacements in the (a) HLW, (b) L/ILW, and (c) shaft models

3.8 Definition of fracture aperture and slip (for output purposes)

In a FDEM model, the simulated fractures are composed of a series of failed crack elements. Plotting of the contours of fracture aperture and slip of a simulated EDZ fracture network is based on the geometrical configuration of a failed crack element depicted in Fig. 3-10. A local xy system of reference is centered in the centroid of the polygon $ABCD$ defined by the two finite element edges constituting the crack element. The orientation of the x -axis is defined by the line connecting the midpoints of lines AD and BC (points G and H). The fracture aperture and slip are defined by the distances between the two mid-edge reference points, E and F , measured along the y - and x -axis.

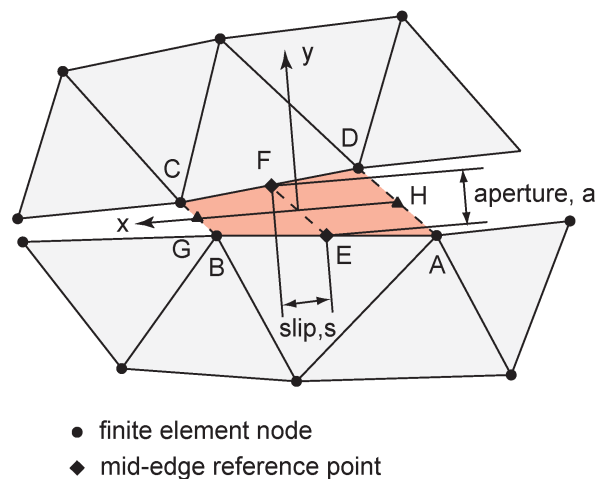


Fig. 3-10: Geometrical configuration of a failed crack element and definition of fracture aperture and slip

4 Results: EDZ simulations for HLW emplacement tunnels

The EDZ was simulated for nine cases of the HLW emplacement tunnel, with varying horizontal in-situ stress, rock strength, core softening ratio, and core softening scheme. For each model, the core softening ratio was adjusted such that the maximum deformation at one of the four monitoring points converges approximately 6.1 cm before the liner is installed. For all models, an initial pore pressure of 9 MPa is applied to the rock mass, with the liner being prescribed zero pore pressure. In the following sub-sections, the results are presented in the form of ground reaction curves, evolution of displacement of select points located on the excavation boundary, fracture patterns, contours of displacement, contours of effective stress (in the rock mass) and total stress (in the lining), and contours of fracture aperture and slip.

Tab. 4-1: Summary of HLW emplacement tunnel simulations performed in this study

Case	Total horizontal stress, σ_{xx} [MPa]	Total vertical stress, σ_{yy} [MPa]	Rock strength properties	Core softening scheme	Core softening ratio, α_{st} [α_{st}]
1	17.5 (σ_h)	22.5	Lower	Isotropic	0.0025
2	17.5 (σ_h)	22.5	Upper	Isotropic	0.0010
3	22.5	22.5	Lower	Isotropic	0.0020
4	22.5	22.5	Upper	Isotropic	0.0015
5	29.25 (σ_H)	22.5	Lower	Isotropic	0.0040
6	29.25 (σ_H)	22.5	Upper	Isotropic	0.0030
7*	17.5 (σ_h)	22.5	Lower	Anisotropic	0.0005 (0.04)
8*	22.5	22.5	Lower	Anisotropic	0.0005 (0.04)
9*	29.25 (σ_H)	22.5	Lower	Anisotropic	0.0005 (0.05)

Note: If only one α_s value is stated, then $\alpha_s = \alpha_{sl} = \alpha_{st}$.

* Results of these cases are shown and discussed in 8A.3.1.

4.1 Minimum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for HLW models with minimum horizontal in-situ stress is shown in Fig. 4-1. The stiffness of the core and liner materials were reduced from its original values over 0.8 M time steps (from 0.3 M to 1.1 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0025$ for lower strength model and $\alpha_s = 0.0010$ for upper strength model) that resulted in approximately 6.1 cm (i.e., 3.3% of the tunnel radius) of convergence at one of the four boundary monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 4-2, as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. In both rock strength cases, the monitoring point that reached approximately 6.1 cm of convergence was P₂₇₀

or the floor of the excavation. Generally, the crown and floor of the excavation converged more than the sidewalls due to the anisotropic stiffness of the rock, and hence excavation core – where the core was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for HLW models with minimum horizontal in-situ stress is shown in Fig. 4-3. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). After installing the stiff liner, there was negligible additional convergence simulated (at most 0.03 cm for the case of lower rock strength).

The extent of the EDZ for the lower bound strength simulation (Fig. 4-3a) are 5.7 m from the sidewalls and 4.5 m from the crown/floor of the excavation (extent of the EDZ defined by the distance from the tunnel wall, where no discontinuities are observed). The overall shape of the EDZ can be described as an octagon that is elongated in the direction of the minor principal stress (i.e., sidewalls). In the crown/floor, fracturing is sparse beyond a depth of 3.4 m from the excavation boundary. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the roof/floor and conjugate shear fractures in the sidewalls, that tend to orient towards the direction of the major principal stress. As these fractures intersect and coalesce, a very dense fracture network emerges and discrete blocks that are approximately one element size (i.e., $h = 0.04$ m) are formed near the excavation. The depth of this dense fracture network is approximately 3.3 m from the sidewall and 2.5 m from the crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 4-3b) are 3.9 m from the sidewalls and 1.7 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as a rectangle that was elongated in the direction of σ_h (i.e., sidewalls). The elongation of the EDZ in the direction of σ_h is a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress are at the sidewalls of the excavation. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. Overall, the simulated fracture network for the upper bound strength case are significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 4-4. As observed in the contour plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement are bedding parallel and intra-layer fractures above/below the crown/floor and conjugate shear fractures in the sidewalls at a depth of 1.0-1.5 m from the excavation boundary. There is greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there is sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 4-5 and Fig. 4-6. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures in the sidewalls. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 4-7). In the rock mass above and below the extent of the EDZ, the effective maximum principal stress (= maximum total principal stress minus pore pressure) was also reduced. In the liner, tangential stresses of up to 8 MPa and 4 MPa in the crown/floor were simulated for the lower and upper rock strength cases.

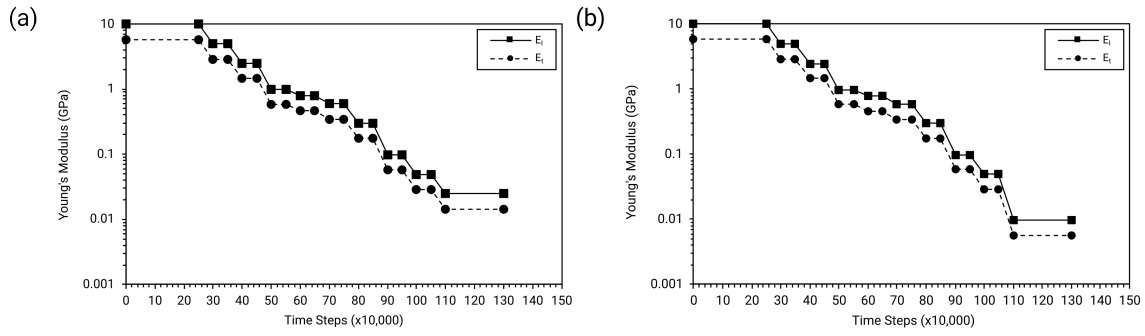


Fig. 4-1: The temporal softening scheme of the excavation core for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

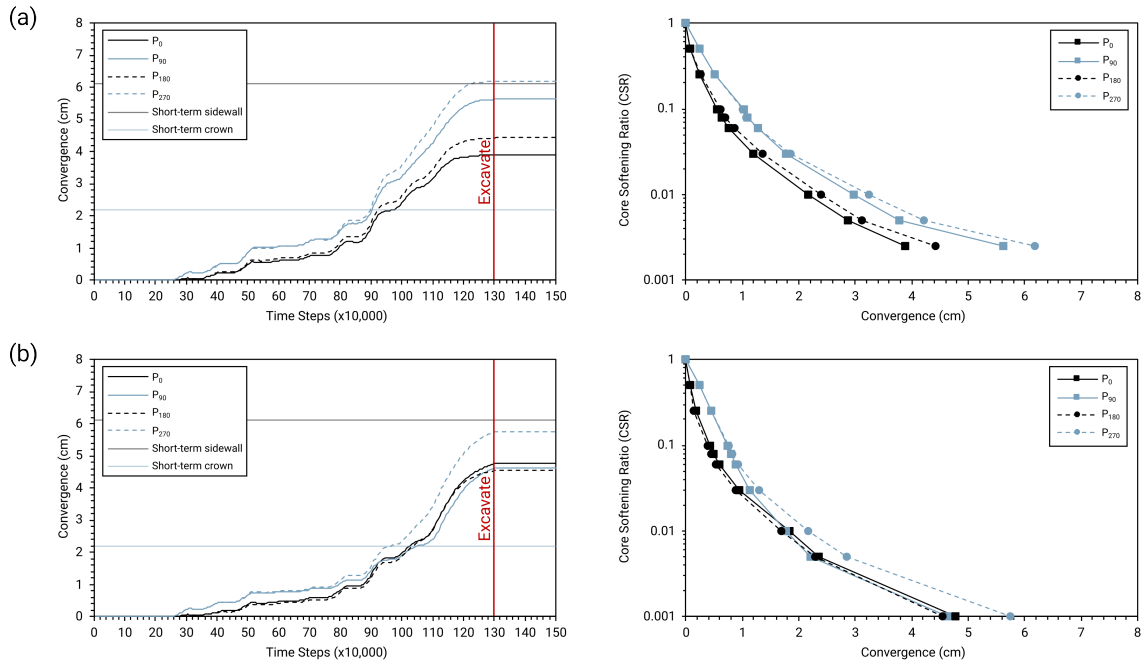


Fig. 4-2: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

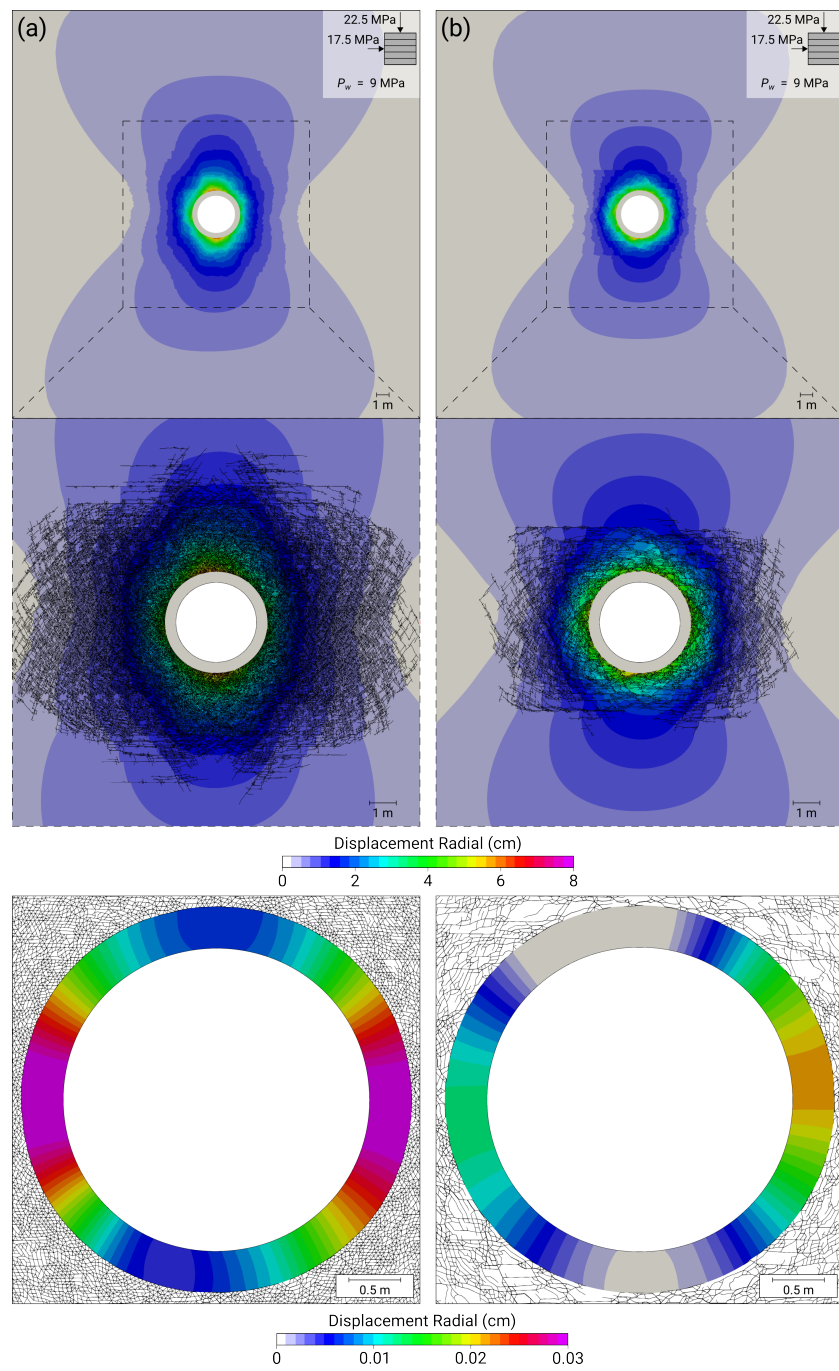


Fig. 4-3: Radial displacement contours and associated EDZ fracture pattern for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

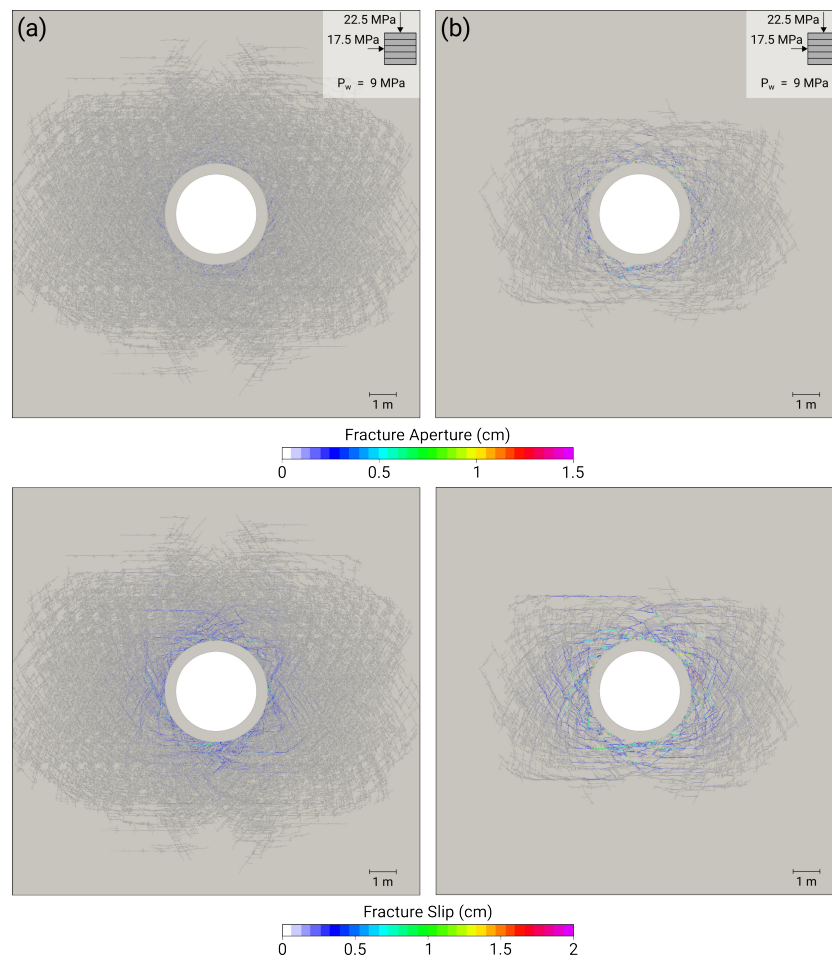


Fig. 4-4: Fracture aperture (top) and slip (bottom) contours for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

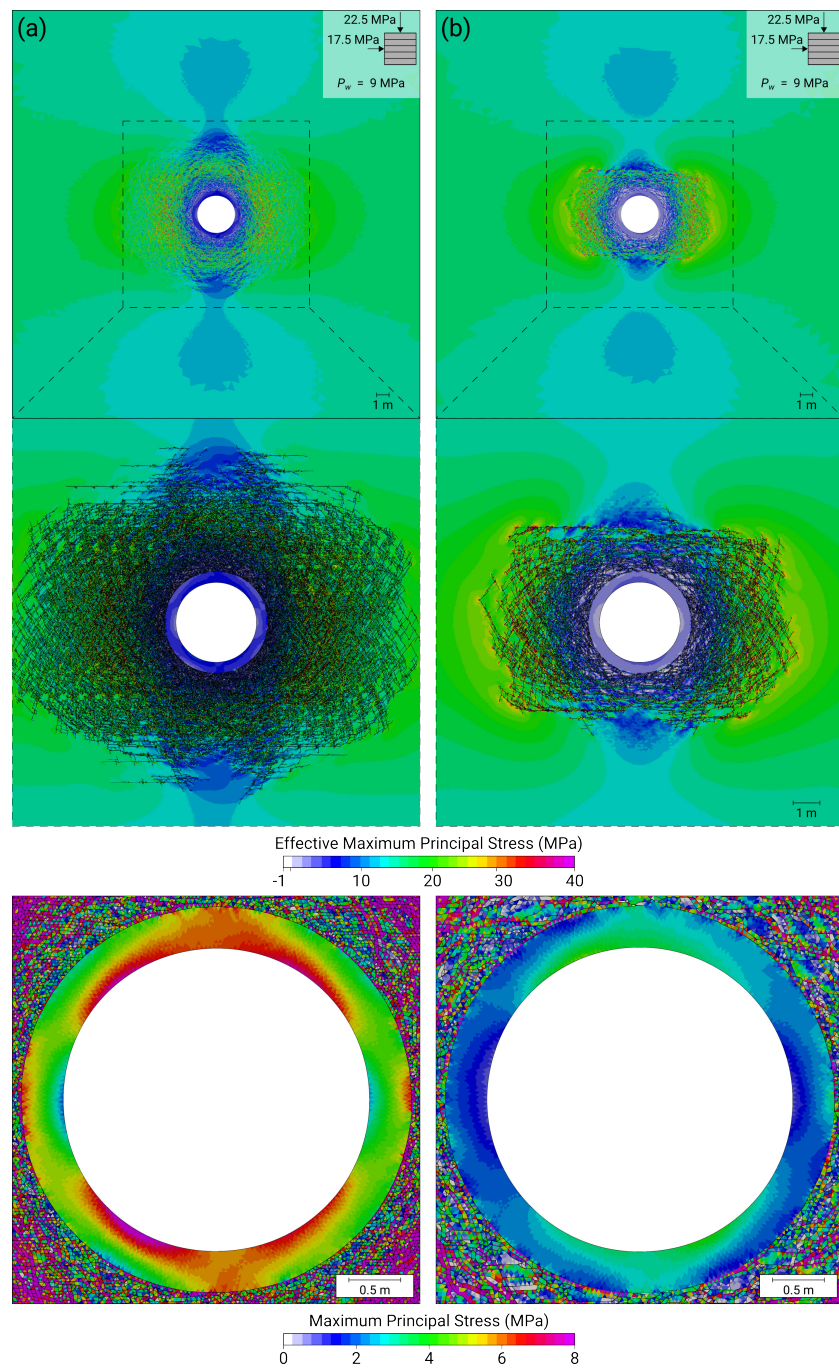


Fig. 4-5: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

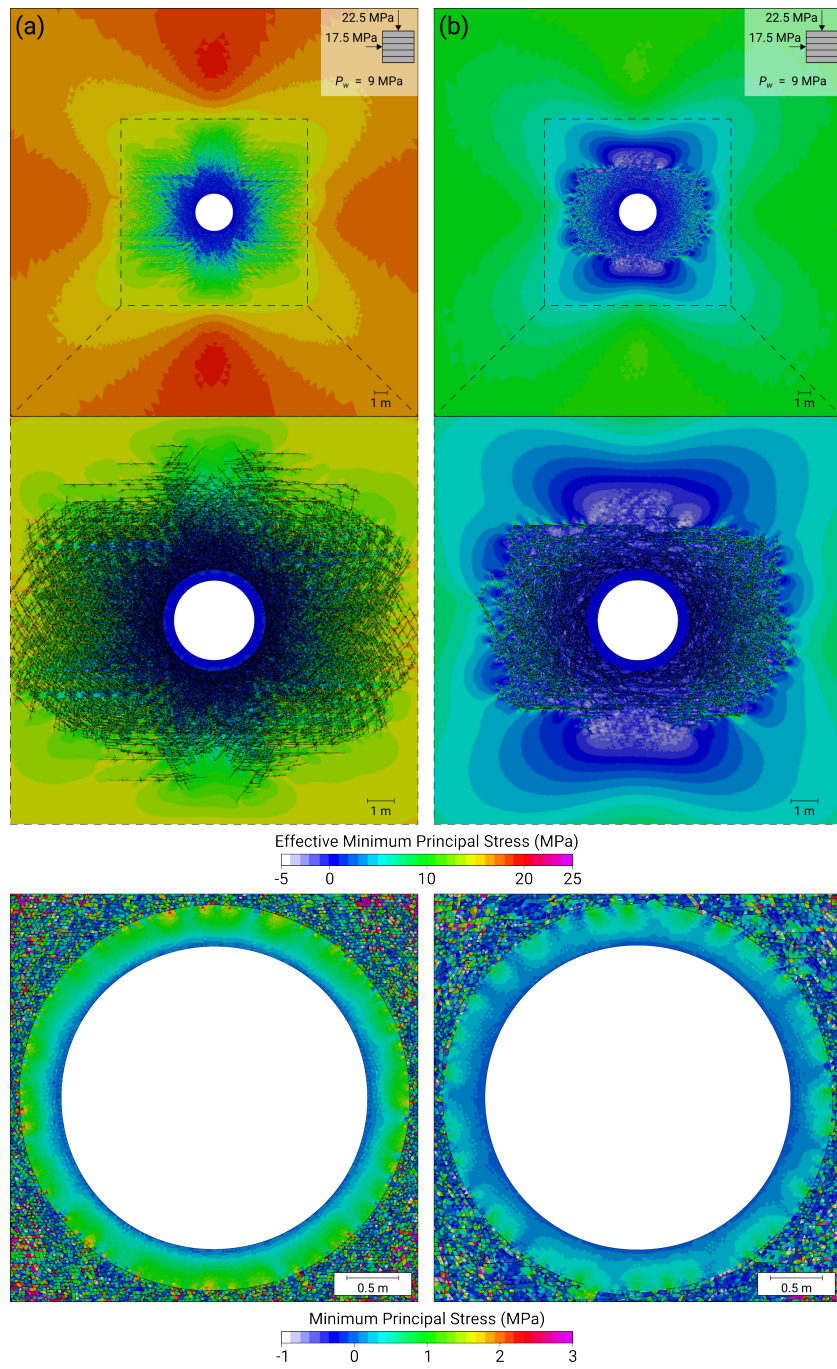


Fig. 4-6: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

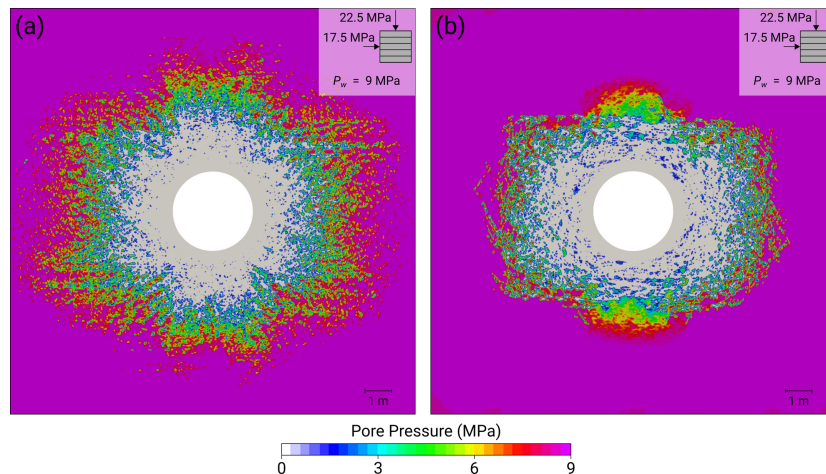


Fig. 4-7: Pore pressure contours for HLW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

4.2 Isotropic in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for HLW models with isotropic in-situ stress is shown in Fig. 4-8 (cases 3 and 4). The stiffness of the core and liner materials were reduced from its original values over 0.8 M time steps (from 0.3 M to 1.1 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0020$ for lower strength model and $\alpha_s = 0.0015$ for upper strength model) that resulted in approximately 6.1 cm (i.e., 3.3% of the tunnel radius) of convergence at one of the four boundary monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 4-9, and as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. For the lower strength case, P₉₀ or the crown, while the upper strength case P₂₇₀ or the floor of the excavation reached the target convergence. Generally, the crown and floor of the excavation converged more than the sidewalls due to the anisotropic stiffness of the rock – where the rock was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for HLW models with an isotropic in-situ stress is shown in Fig. 4-10. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result. After installing the stiff liner, there was negligible additional convergence simulated (at most 0.03 cm for the case of lower rock strength).

The extent of the EDZ for the lower bound strength simulation (Fig. 4-10a) is 4.8 m from the sidewalls and 5.3 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as an octagon. The EDZ has a fairly isotropic depth all around the excavation and its overall shape is largely dictated by the applied isotropic stress field. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersect and coalesce, a dense fracture network emerged and discrete blocks that are approximately one element size (i.e., $h = 0.04$ m) are formed near the excavation. The depth of this dense fracture network is approximately 3.3 m from the sidewall and 3.5 m from the crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 4-10b) is 3.3 m from the sidewalls and 3.1 m from the crown/floor of the excavation. The overall shape of the EDZ is a square around the excavation. The EDZ has a fairly isotropic depth all around the excavation and its overall shape is largely dictated by the applied isotropic in-situ stress field. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. Overall, the simulated fracture network for the upper bound strength case is significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations is shown in Fig. 4-11. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement are bedding parallel and intra-layer fractures above/below the crown/floor and conjugate shear fractures in the sidewalls. This region is at an approximate depth of 1.3 m from the excavation boundary.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 4-12 and Fig. 4-13. Regions of greatest compressive stress are at the fracture tips of conjugate shear fractures and bedding parallel fractures at the extents of the EDZ. There are regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. In addition, within this region, due to the increase in aperture from fracturing the pore pressure has effectively reduced from 9 MPa to zero (Fig. 4-14). In the liner, tangential stresses of up to 7.5 MPa and 5 MPa are simulated in the crown/floor for lower and upper rock strength cases.

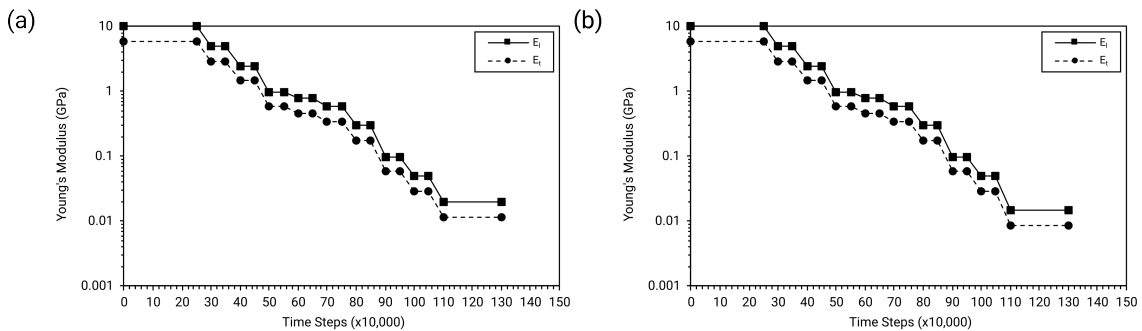


Fig. 4-8: The temporal softening scheme of the excavation core for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

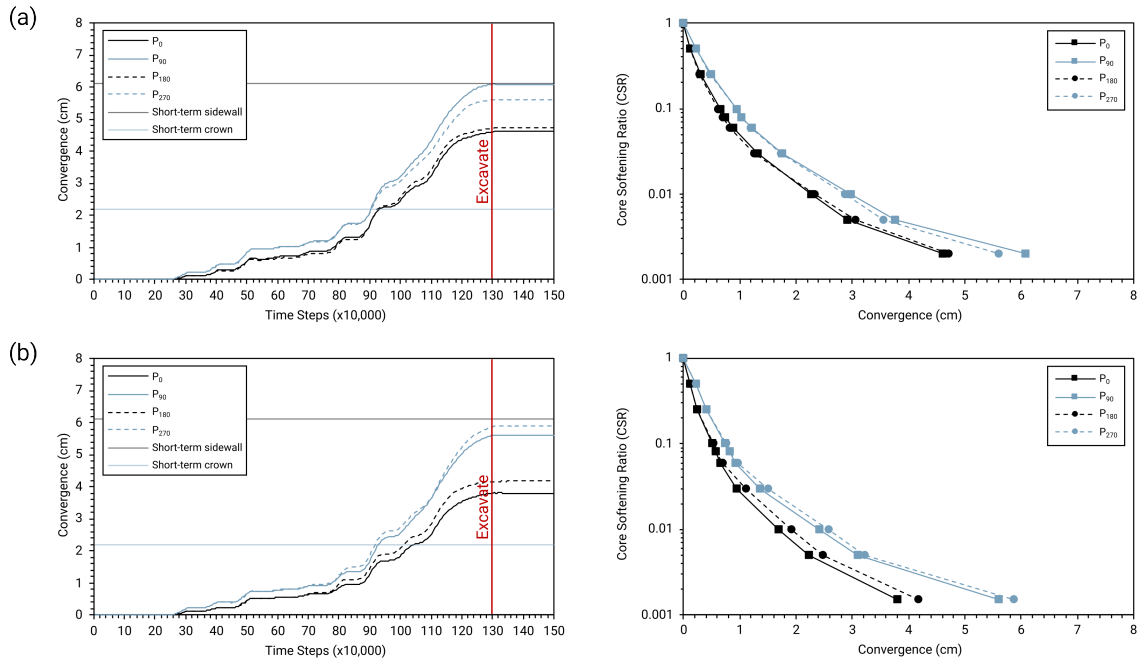


Fig. 4-9: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

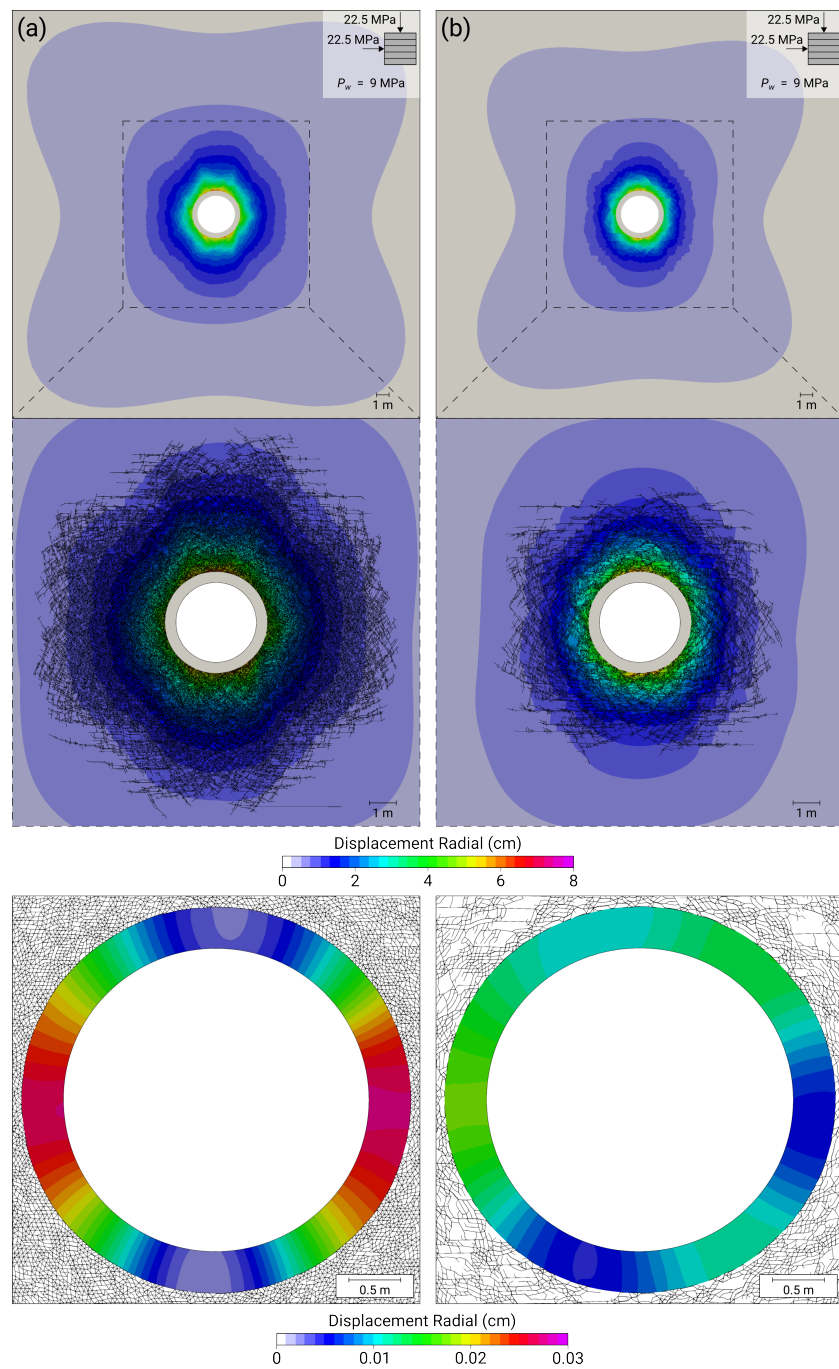


Fig. 4-10: Radial displacement contours and associated EDZ fracture pattern for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

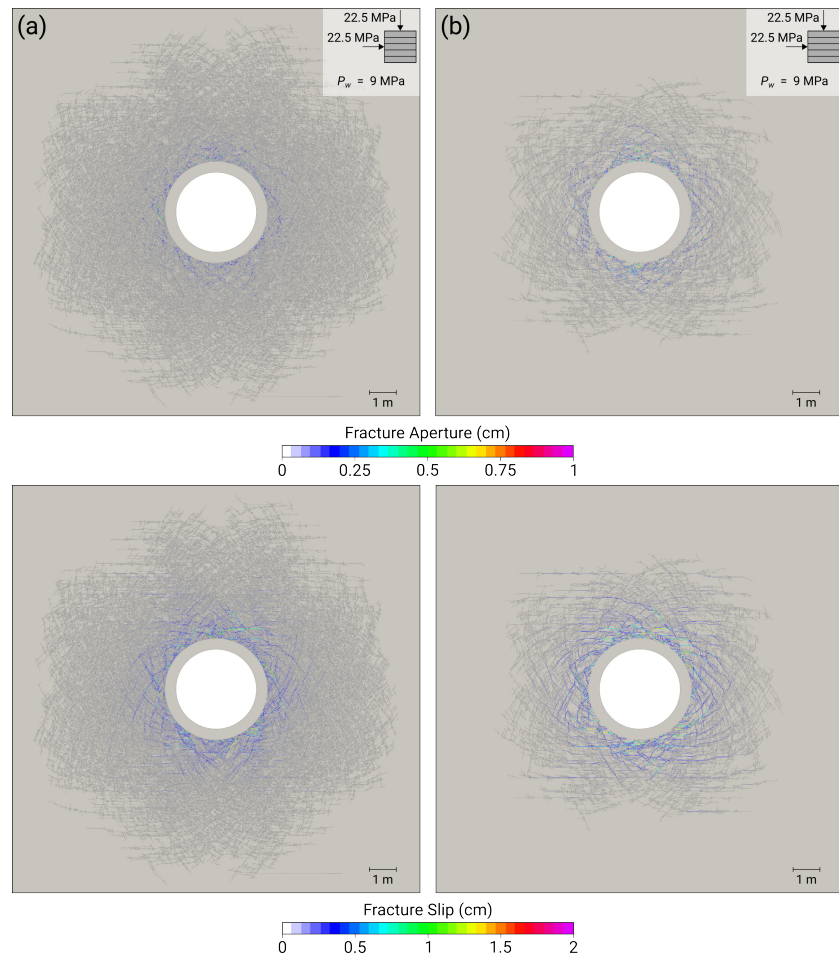


Fig. 4-11: Fracture aperture (top) and slip (bottom) contours for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

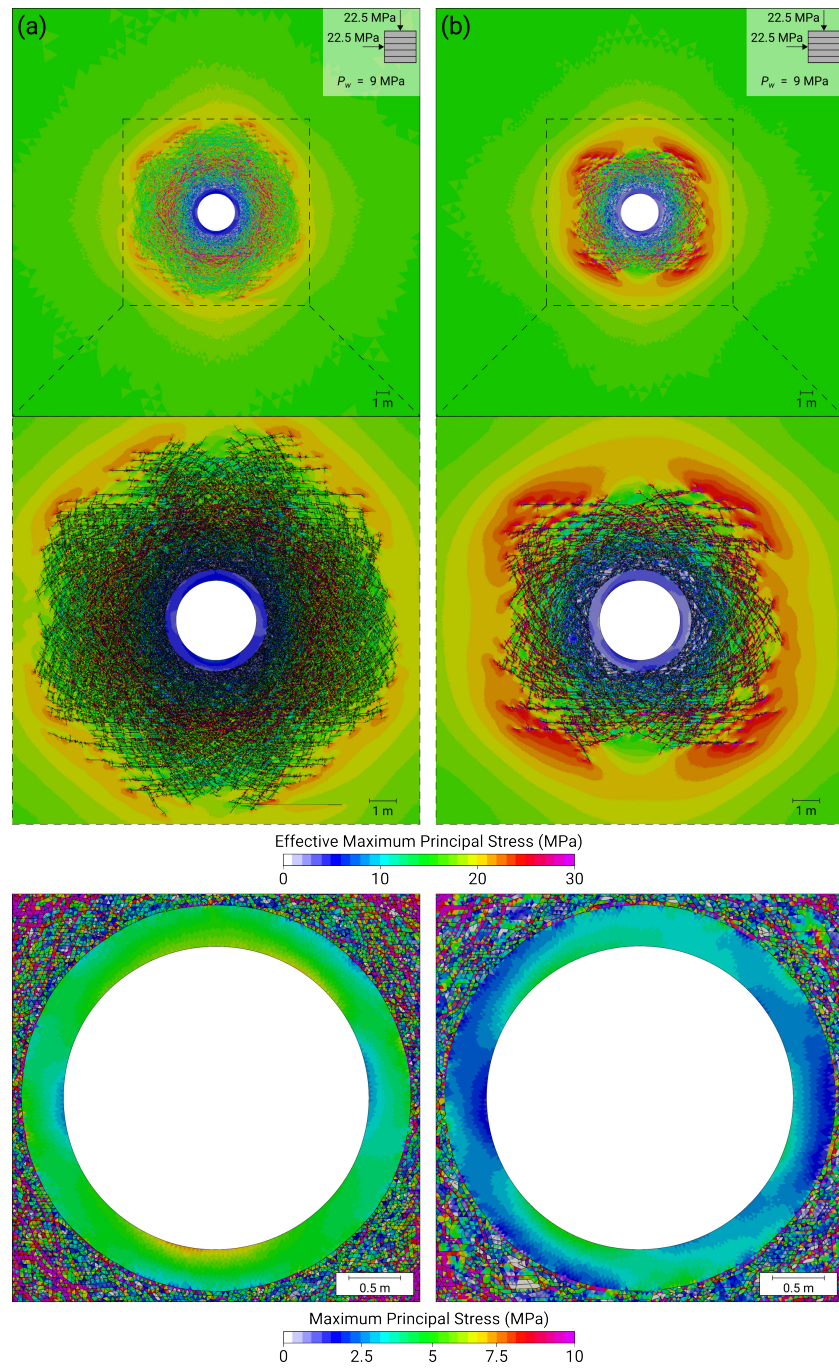


Fig. 4-12: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

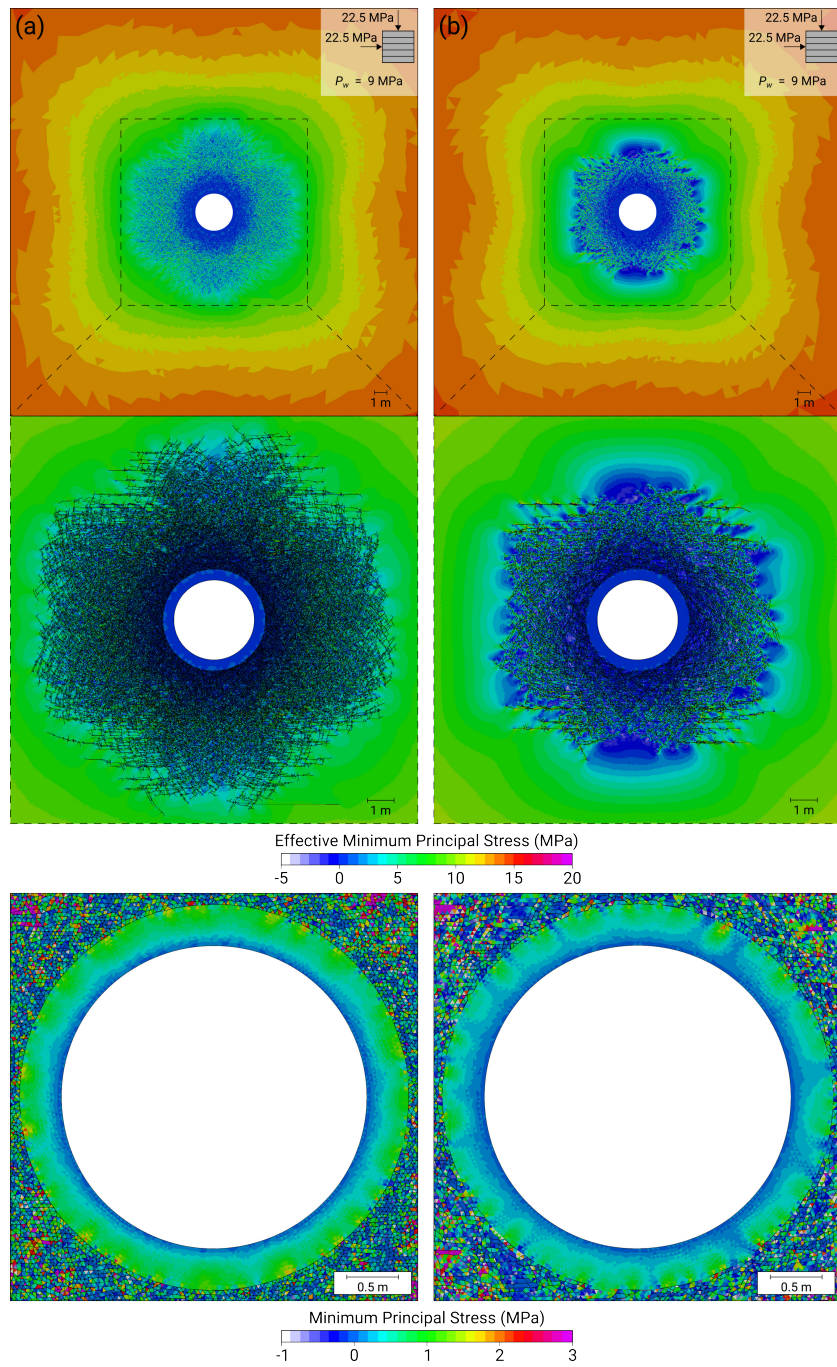


Fig. 4-13: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for HLW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

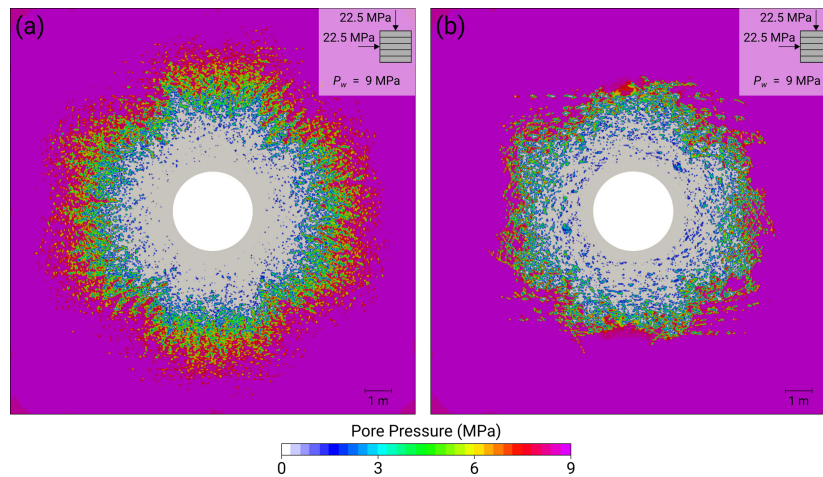


Fig. 4-14: Pore pressure contours for HLW tunnel models with isotropic in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations.

4.3 Maximum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for HLW models with maximum horizontal in-situ stress is shown in Fig. 4-15 (cases 5 and 6). The stiffness of the core and liner materials were reduced from 0.3 M to 1.1 M time steps where it was held at the corresponding core softening ratio ($\alpha_s = 0.0040$ for lower strength model and $\alpha_s = 0.0030$ for upper strength model) that resulted in approximately 6.1 cm (i.e., 3.3% of the tunnel radius) of convergence at one of the four boundary monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 Mio time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 4-16, and as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. For both strength cases, P_{90} or the crown of the excavation reached the target convergence. Generally, the crown and floor of the excavation converged more than the sidewalls due to the anisotropic stiffness of the rock – where the rock was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for HLW models with the maximum horizontal in-situ stress is shown in Fig. 4-17. The liner displacement shown was the difference in convergence from the time of liner installation. After installing the stiff liner, there was negligible additional convergence simulated (at most 0.05 cm for the case of lower rock strength). The overall shape of the EDZs could be described as octagons that were elongated in the direction of the minor in-situ stress (i.e., crown/floor). The elongation of the EDZ in the direction of σ_v was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the crown/floor of the excavation.

The extent of the EDZ for the lower bound strength simulation (Fig. 4-17a) are 3.2 m from the sidewalls and 5.0 m from the crown/floor of the excavation. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the roof/floor and conjugate shear fractures in the sidewalls. As these fractures intersect and coalesce, a very dense fracture network emerges and discrete blocks that are approximately one element size (i.e., $h = 0.04$ m) are formed

near the excavation. The depth of this dense fracture network is approximately 3.3 m from the sidewall and 2.1 m from the crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 4-17b) are 1.7 m from the sidewalls and 3.1 m from the crown/floor of the excavation. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the roof/floor and conjugate shear fractures in the sidewalls. Overall, the simulated fracture network for the upper bound strength case is significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 4-4. As observed in the plot, the region near the excavation wall had the greatest aperture and slip. Regions of significant shear displacement were bedding parallel and intra-layer fractures above/below the crown/floor, and to a lesser extent conjugate shear fractures in the sidewall.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 4-19 and Fig. 4-20. Regions of greatest compressive stress were at the fracture tips of bedding parallel and intra-layer fractures in the crown/floor of the excavation. This region correlated with high compressive stresses as a result of the stress redistribution of the maximum horizontal far-field stresses around the excavation. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. In addition, within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 4-21). In the liner, tangential stresses of up to 12 MPa and 9 MPa were simulated in the crown/floor for lower and upper rock strength cases.

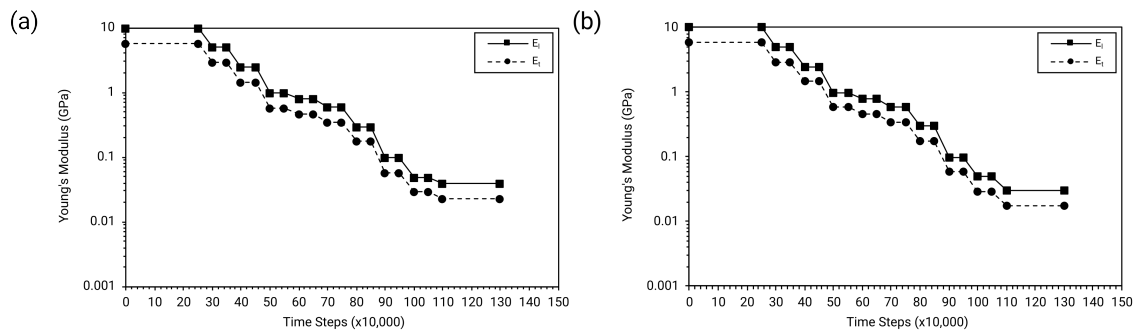


Fig. 4-15: The temporal softening scheme of the excavation core for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

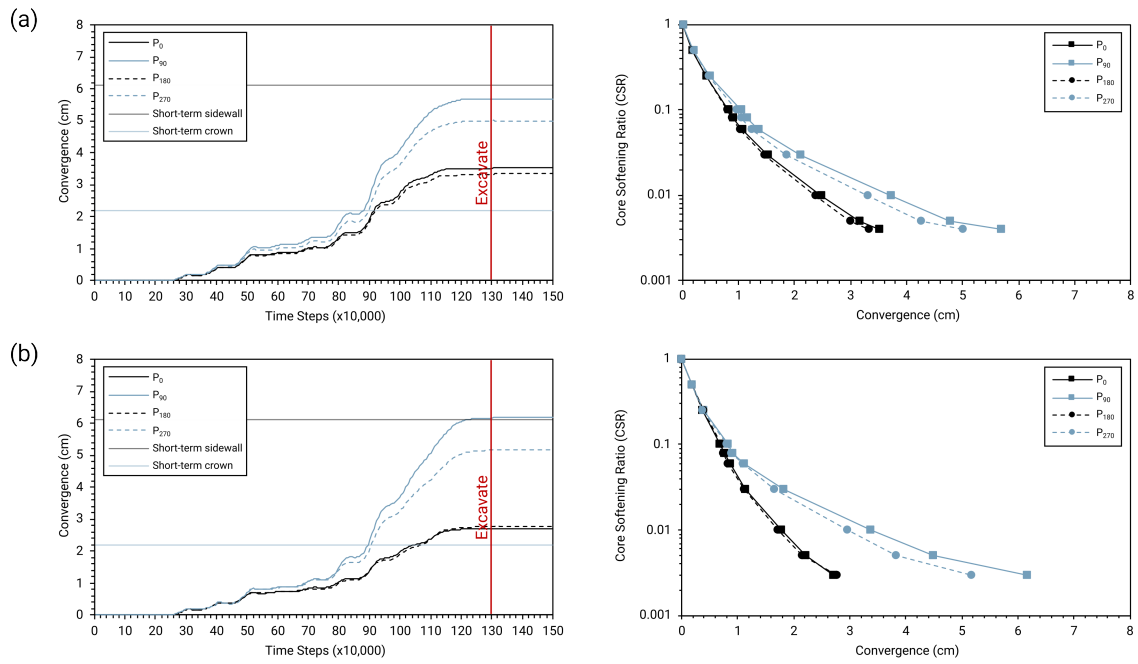


Fig. 4-16: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

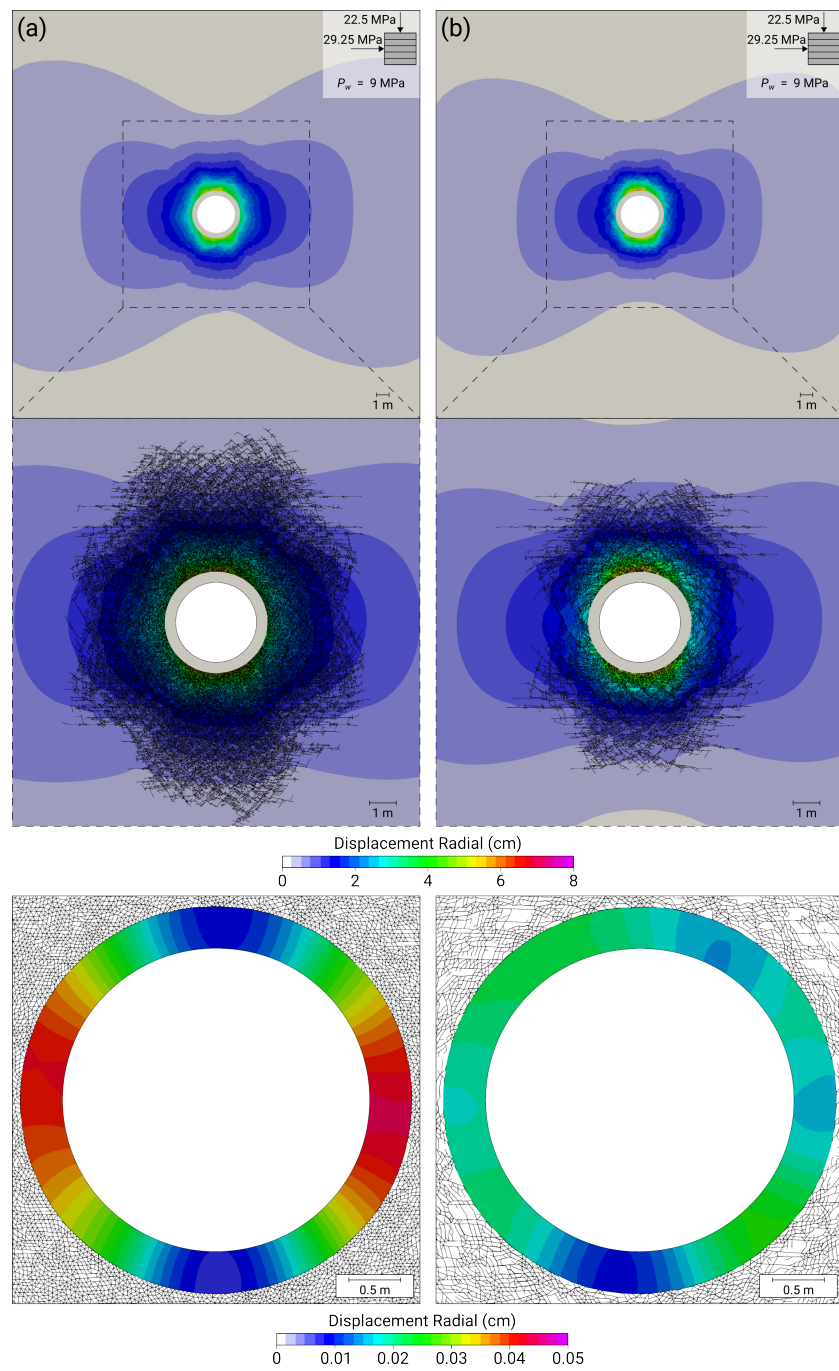


Fig. 4-17: Radial displacement contours and associated EDZ fracture pattern for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

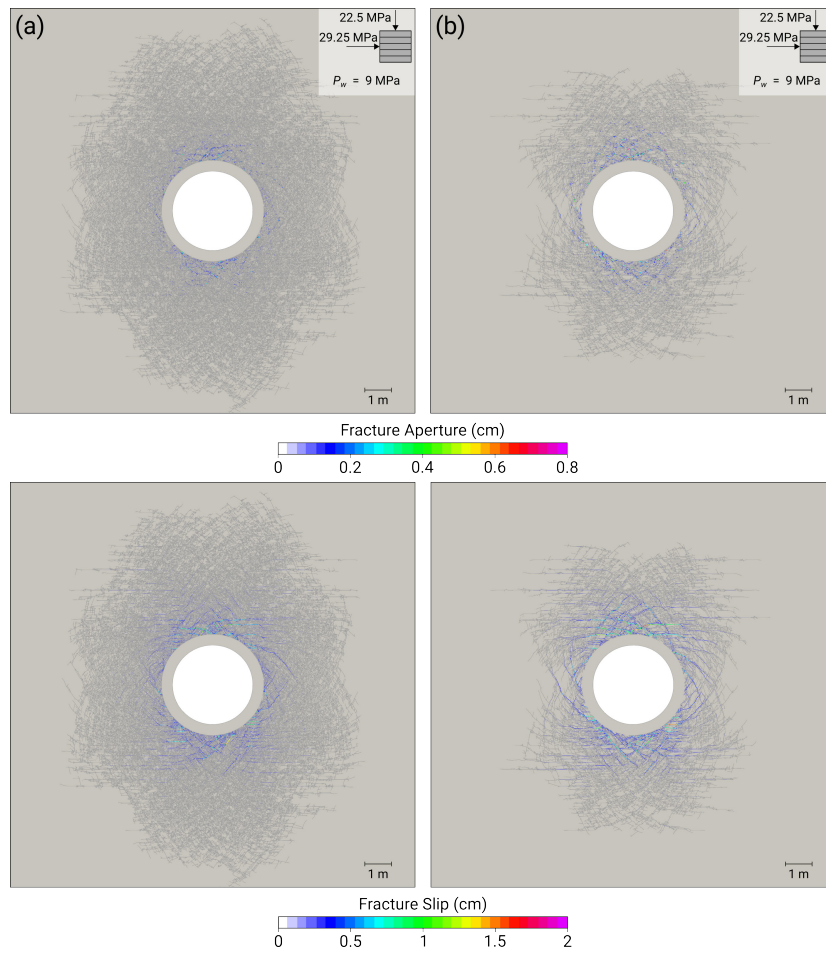


Fig. 4-18: Fracture aperture (top) and slip (bottom) contours for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

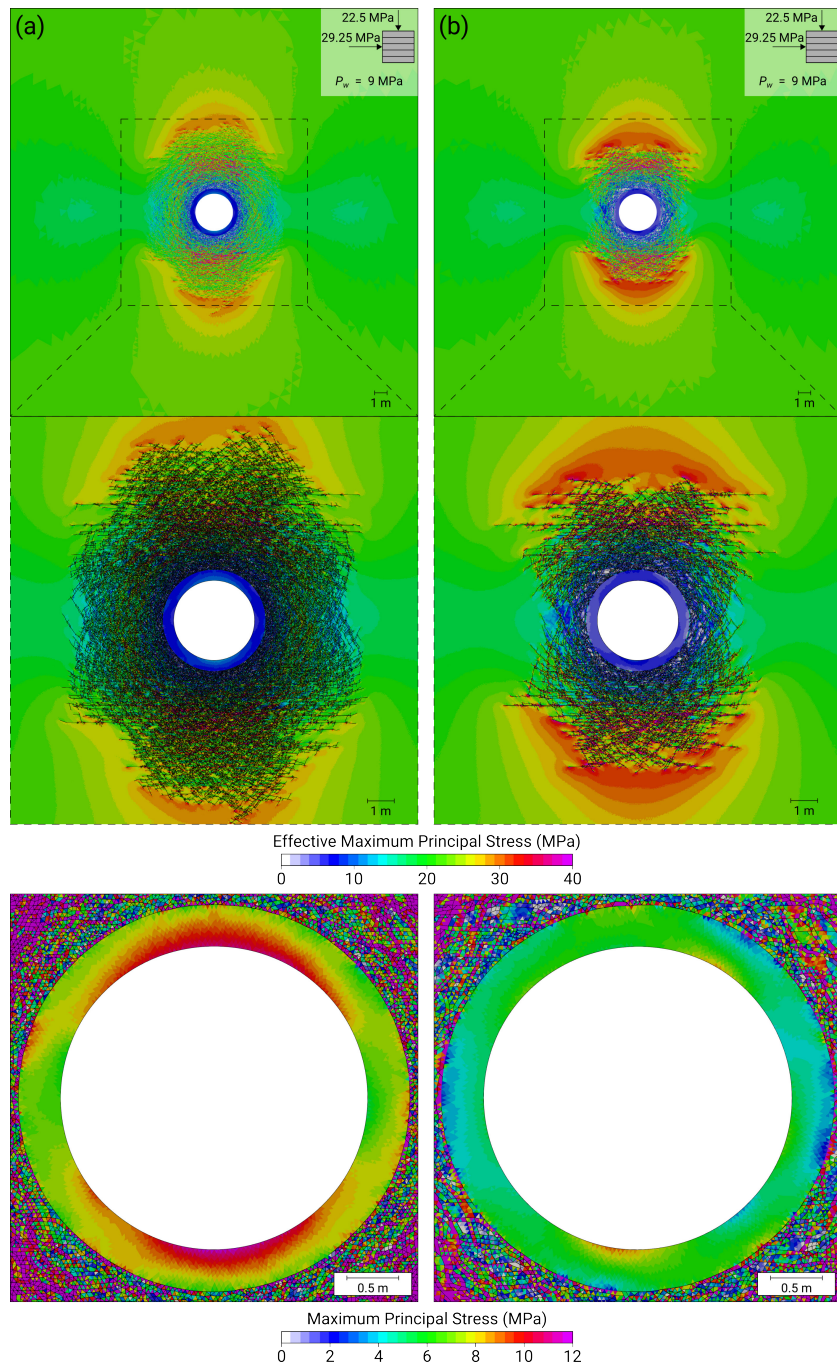


Fig. 4-19: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

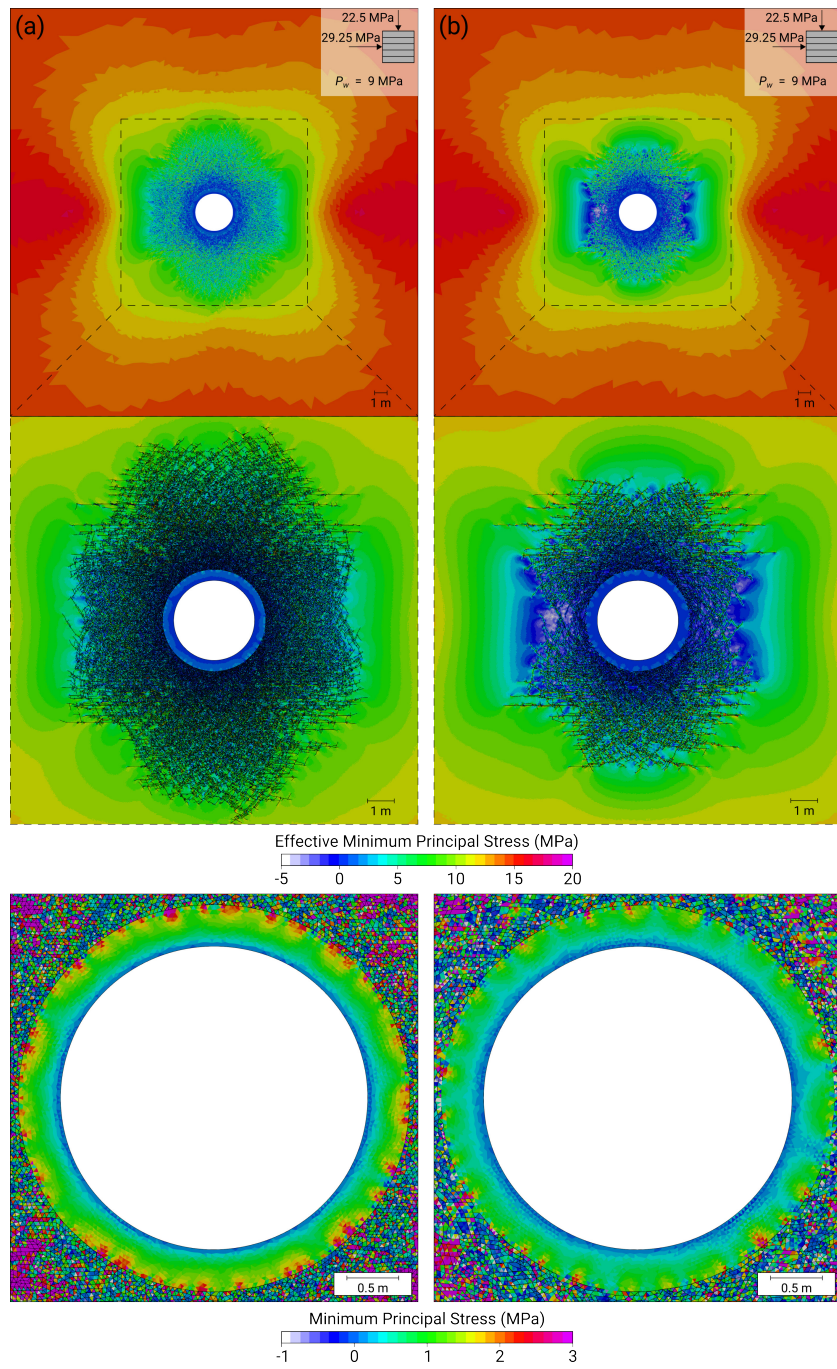


Fig. 4-20: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

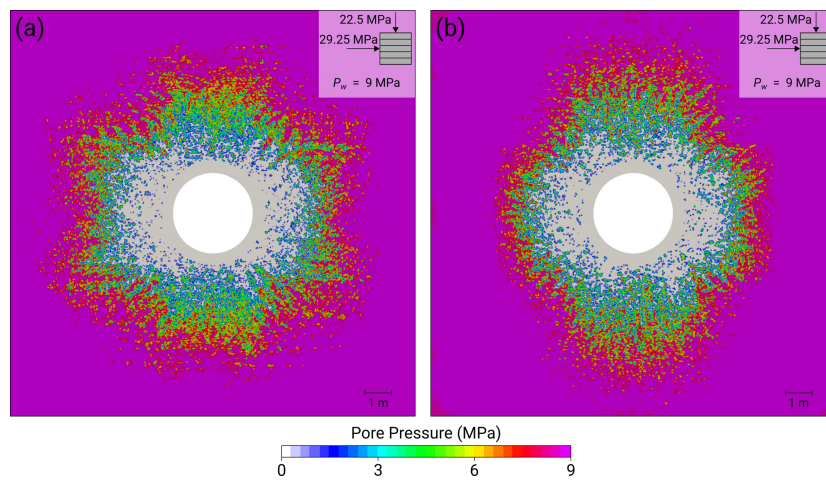


Fig. 4-21: Pore pressure contours for HLW tunnel models with maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations.

5 Results: EDZ simulations for L/ILW emplacement cavern

The EDZ was simulated for nine cases of the L/ILW emplacement cavern, with varying horizontal in-situ stress, rock strength, core softening ratio, and core softening scheme. For each model, the core softening ratio was adjusted such that the maximum deformation at one of the four monitoring points converges approximately 20.5 cm before the liner is installed. For all models, an initial pore pressure of 9 MPa is applied to the rock mass, with the liner being prescribed zero pore pressure. In the following sub-sections, the results are presented in the form of ground reaction curves, evolution of displacement of select points located on the excavation boundary, fracture patterns, contours of displacement, contours of effective stress (in the rock mass) and total stress (in the lining), and contours of fracture aperture and slip.

Tab. 5-1: Summary of L/ILW emplacement tunnel simulations performed in this study

Case	Total horizontal stress, σ_{xx} (MPa)	Total vertical stress, σ_{yy} (MPa)	Rock strength properties	Core softening scheme	Core softening ratio, α_{st} (α_{st})
1	17.5 (σ_h)	22.5	lower	isotropic	0.0025
2	17.5 (σ_h)	22.5	upper	isotropic	0.0010
3	22.5	22.5	lower	isotropic	0.0030
4	22.5	22.5	upper	isotropic	0.00165
5	29.25 (σ_H)	22.5	lower	isotropic	0.0040
6	29.25 (σ_H)	22.5	upper	isotropic	0.00375
7*	17.5 (σ_h)	22.5	lower	anisotropic	0.0005 (0.04)
8*	22.5	22.5	lower	anisotropic	0.0005 (0.04)
9*	29.25 (σ_H)	22.5	lower	anisotropic	0.0008 (0.06)

Note: If only one α_s value is stated, then $\alpha_s = \alpha_{sl} = \alpha_{st}$.

* Results of these cases are shown and discussed in Appendix 8A.3.2.

5.1 Minimum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for L/ILW cavern models with minimum horizontal in-situ stress is shown in Fig. 5-1 (cases 1 and 2). The stiffness of the core and liner materials were reduced from its original values over 0.8 M time steps (from 0.3 M to 1.1 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0025$ for lower strength model and $\alpha_s = 0.0010$ for upper strength model) that resulted in approximately 20.5 cm (i.e., 3.3% of half the excavation span) of convergence at one of the four monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 5-2, and as expected, the walls of the excavation converged as the Young's modulus of the core and liner were reduced. For both strength cases, P₂₇₀ or the floor of the excavation reached the target convergence. Generally, the crown and floor of the excavation converged more than the sidewalls due to the

anisotropic stiffness of the rock, and hence excavation core – where the core was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for L/ILW cavern models with minimum horizontal in-situ stress is shown in Fig. 5-3. The liner displacement shown was the difference in convergence from the time of liner installation. After installing the stiff liner, there was 0.75-1.0 cm of displacement at the sidewalls and 0.25-0.5 cm of liner displacement in the crown and floor for the case of lower bound rock strength. For the upper bound strength case, simulated liner displacement was approximately half of the lower bound strength case. As expected, there was greater liner displacement for the case of lower rock strength due to significantly more fracturing and displacement in the rock mass that the liner was supporting.

The extent of the EDZ for the lower bound strength simulation (Fig. 5-3a) was 19.9 m from the sidewalls and 13.7 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as an octagon that was elongated in the direction of σ_h (i.e., sidewalls). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the sidewalls of the excavation. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.15$ m) were formed near the excavation. The depth of this dense fracture network was approximately 7.4 m from the sidewall and 9.6 m from the crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 5-3b) was 15.9 m from the sidewalls and 8.1 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as a rectangle that was elongated in the direction of σ_h (i.e., sidewalls). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the sidewalls of the excavation. The simulated EDZ contained relatively minor bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and significant conjugate shear fracturing in the sidewalls. Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 5-4. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement are bedding parallel fractures and intra-layer fractures above/below the crown/floor at a depth of 3.6 m. There was greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there was sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 5-5 and Fig. 5-6. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures around the excavation. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 5-7). In the rock mass above and below the extent of the EDZ, the effective maximum principal stress was also reduced. In the liner, tangential stresses of up to 40 MPa and 30 MPa were concentrated at the corners of the liner for the case of lower and upper rock strength. In the sidewall and haunch of the liner, tangential stresses up to 30 MPa and 20 MPa were simulated for lower and upper rock strength cases.

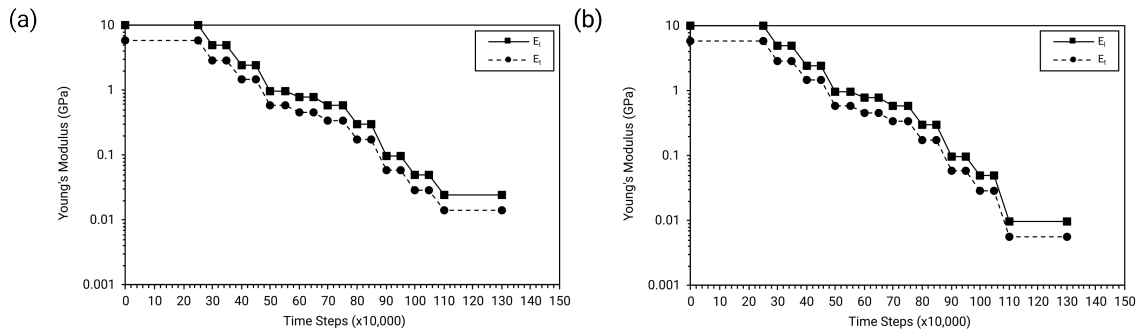


Fig. 5-1: The temporal softening scheme of the excavation core for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

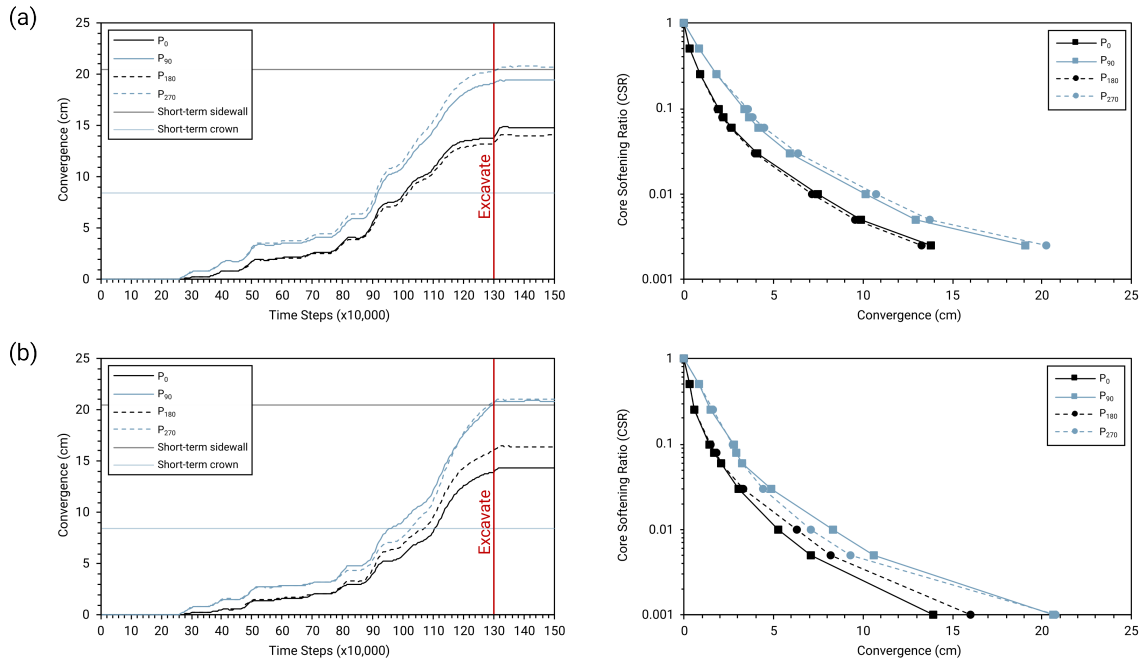


Fig. 5-2: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

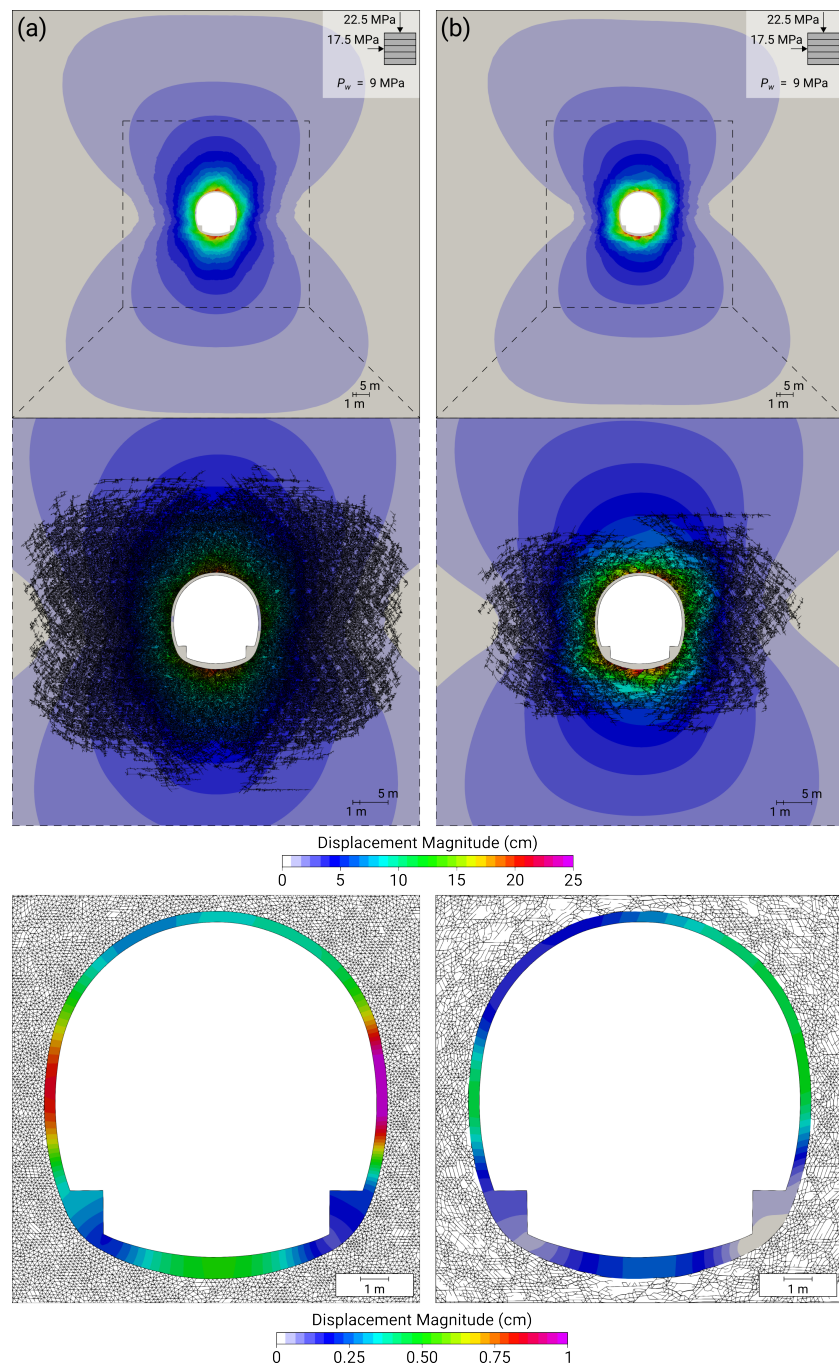


Fig. 5-3: Radial displacement contours and associated EDZ fracture pattern for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

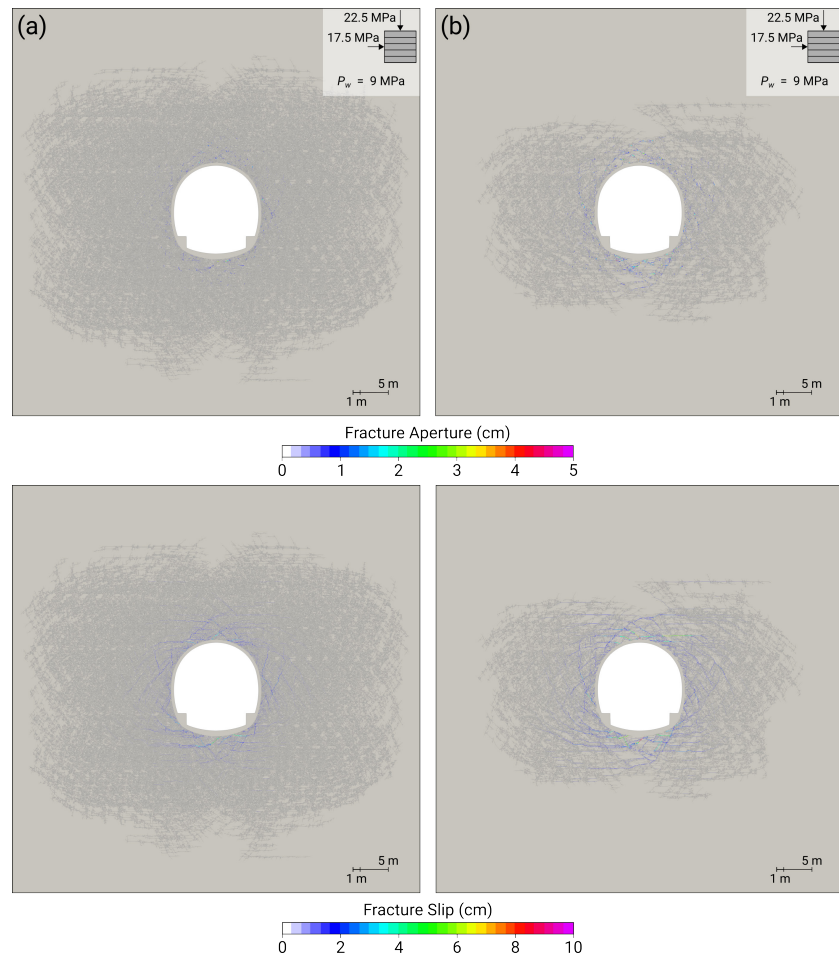


Fig. 5-4: Fracture aperture (top) and slip (bottom) contours for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

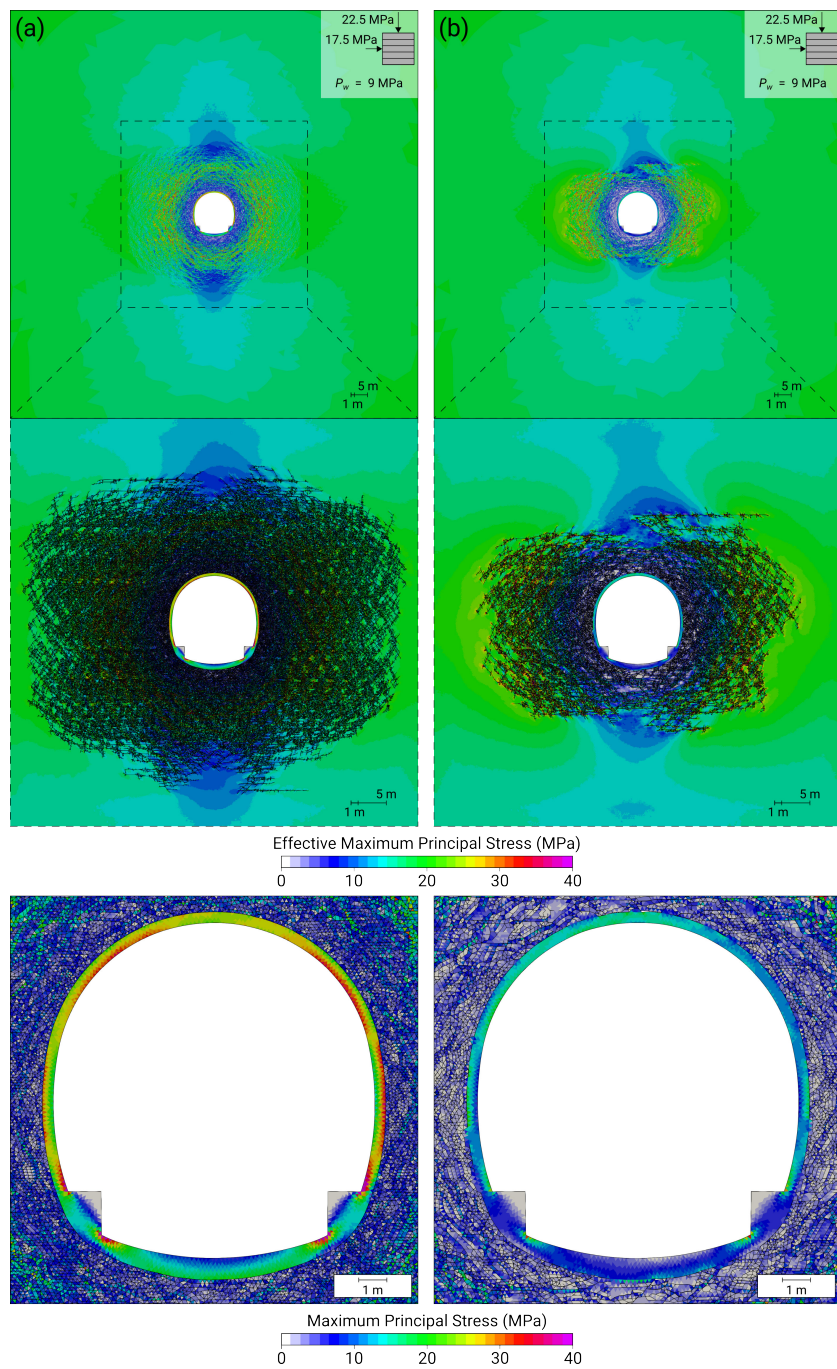


Fig. 5-5: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

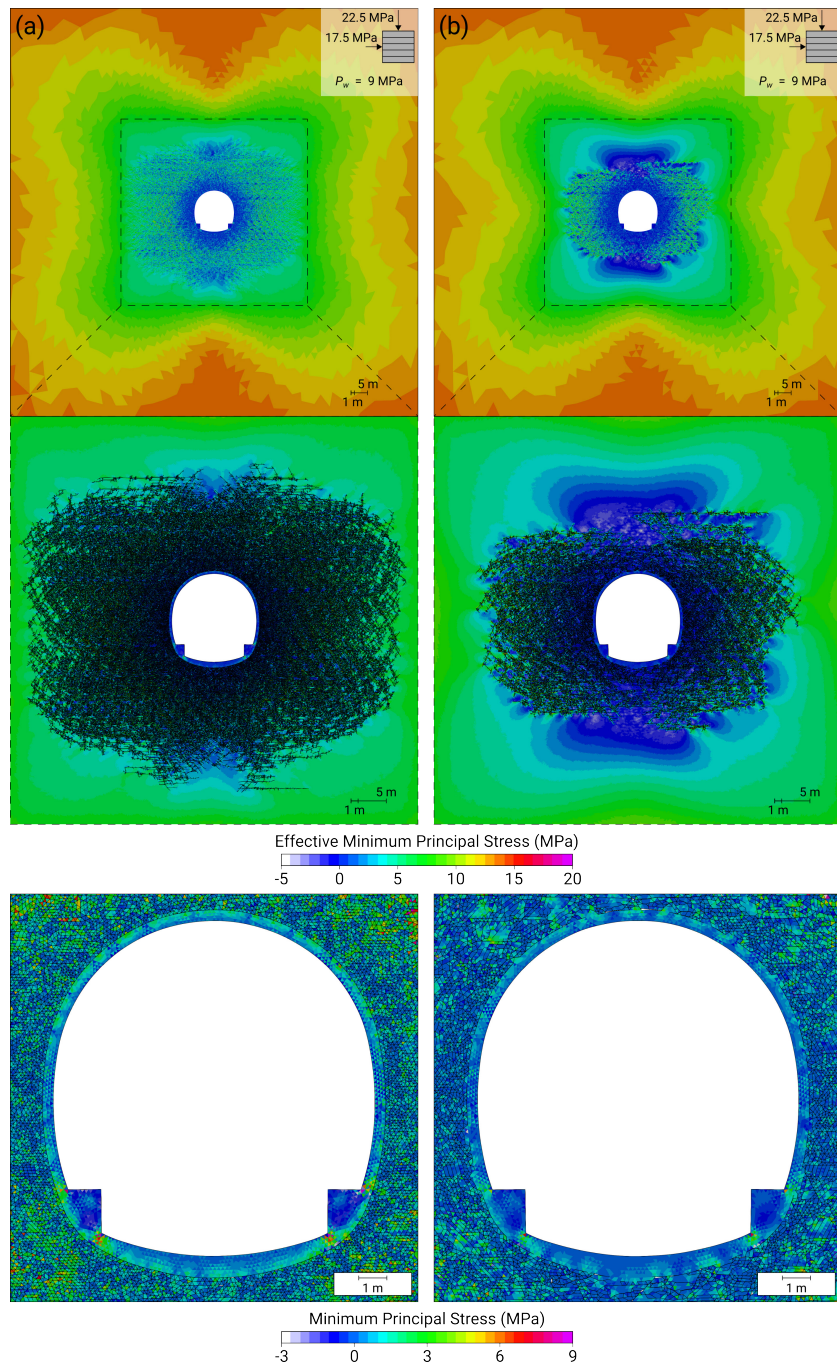


Fig. 5-6: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for L/ILW tunnel models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

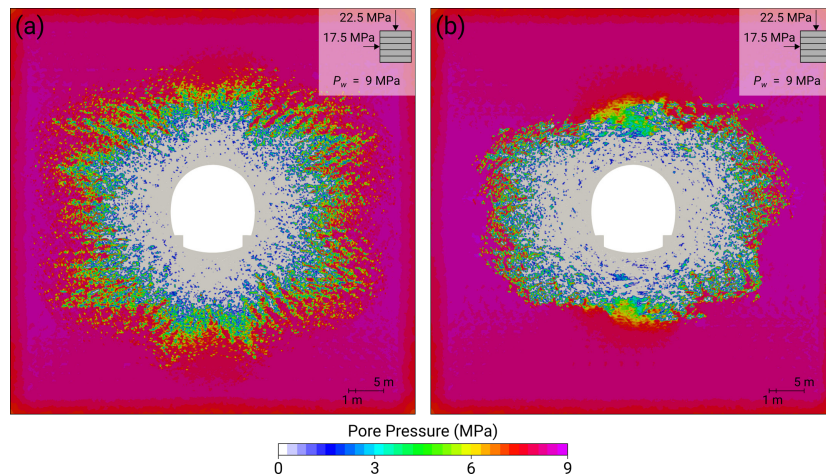


Fig. 5-7: Pore pressure contours for L/ILW cavern models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

5.2 Isotropic in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for L/ILW cavern models with isotropic in-situ stress is shown in Fig. 5-8 (cases 3 and 4). The stiffness of the core and liner materials were reduced from its original values over 0.8 M time steps (from 0.3 M to 1.1 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0030$ for lower strength model and $\alpha_s = 0.00165$ for upper strength model) that resulted in approximately 20.5 cm (i.e., 3.3% of half the excavation span) of convergence at one of the four monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 5-9, and as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. For the lower strength case, P_{90} or the crown, while the upper strength case P_{270} or the floor of the excavation reached the target convergence. Generally, the crown and floor of the excavation converged more than the sidewalls due to the anisotropic stiffness of the rock – where the rock was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for L/ILW cavern models with an isotropic in-situ stress is shown in Fig. 5-10. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result. After installing the stiff liner, there was 0.75-1.0 cm of liner displacement at the sidewalls, and 0.25-0.5 cm of liner displacement in the crown and floor for the case of lower bound rock strength. For the upper bound strength case, simulated liner displacement was approximately half of the lower bound strength case. As expected, there was greater liner displacement for the case of lower rock strength due to significantly more fracturing in the rock mass that the liner is supporting.

The extent of the EDZ for the lower bound strength simulation (Fig. 5-10a) was 18.0 m from the sidewalls and 17.6 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as an octagon. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersect and coalesce, a very dense fracture network emerges and discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) are formed near the excavation.

The depth of this dense fracture network is approximately 10.0 m from the sidewall and crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 5-10b) was 14.8 m from the sidewalls and 13.2 m from the crown/floor of the excavation. The overall shape of the EDZ was an octagon around the excavation. The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 5-11. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement are bedding parallel fractures and intra-layer fractures at a depth of 3.5-4.5 m above/below the crown/floor. Less significant shear displacement was simulated for conjugate shear fractures 4.5-5.5 m into the sidewalls. There was greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there was sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 5-12 and Fig. 5-13. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures and bedding parallel fractures at the extents of the EDZ. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 5-14). In the liner, tangential stresses of up to 45 MPa and 35 MPa were concentrated at the corners of the liner for the case of lower and upper rock strength. In the sidewall and haunch of the liner, tangential stresses up to 35 MPa and 25 MPa were simulated for lower and upper rock strength cases.

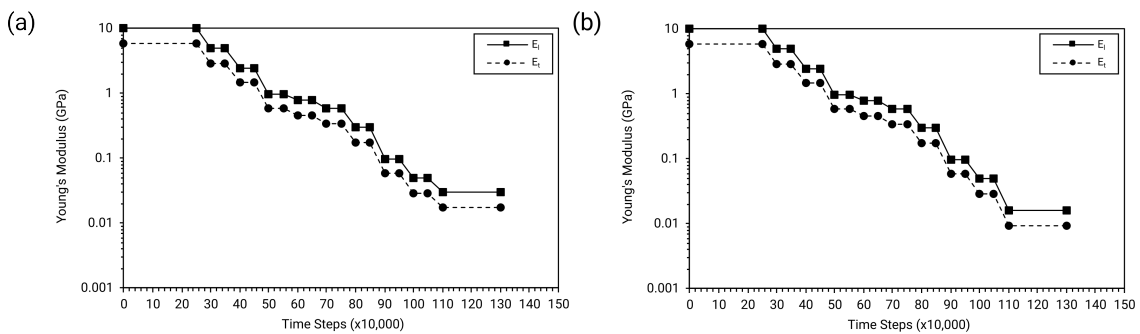


Fig. 5-8: The temporal softening scheme of the excavation core for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

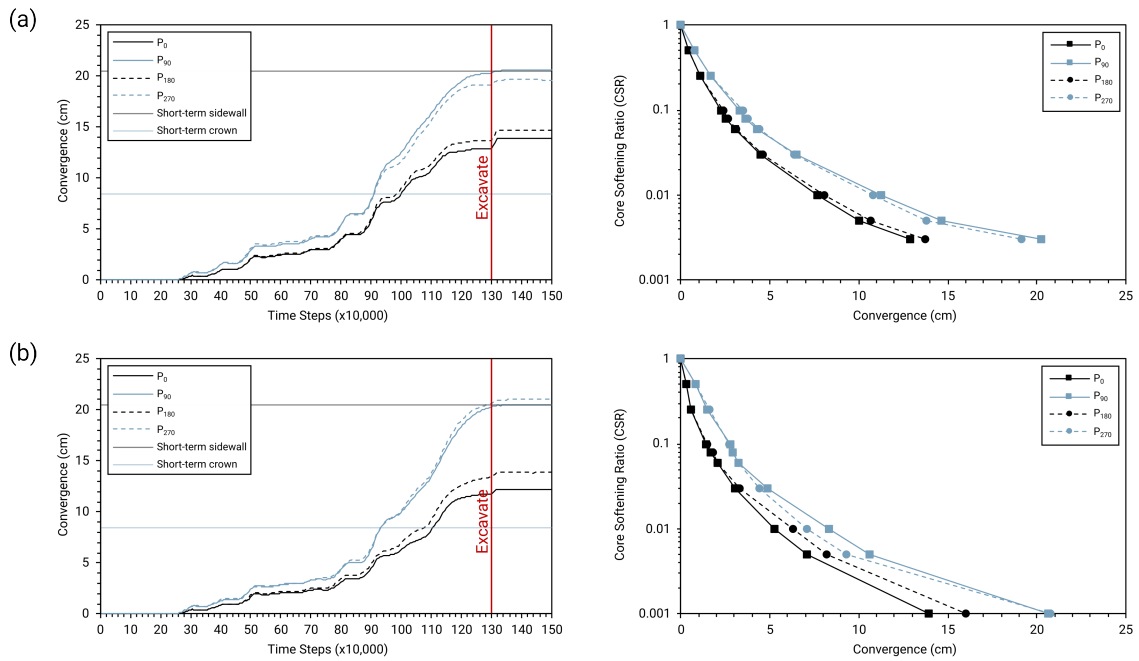


Fig. 5-9: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

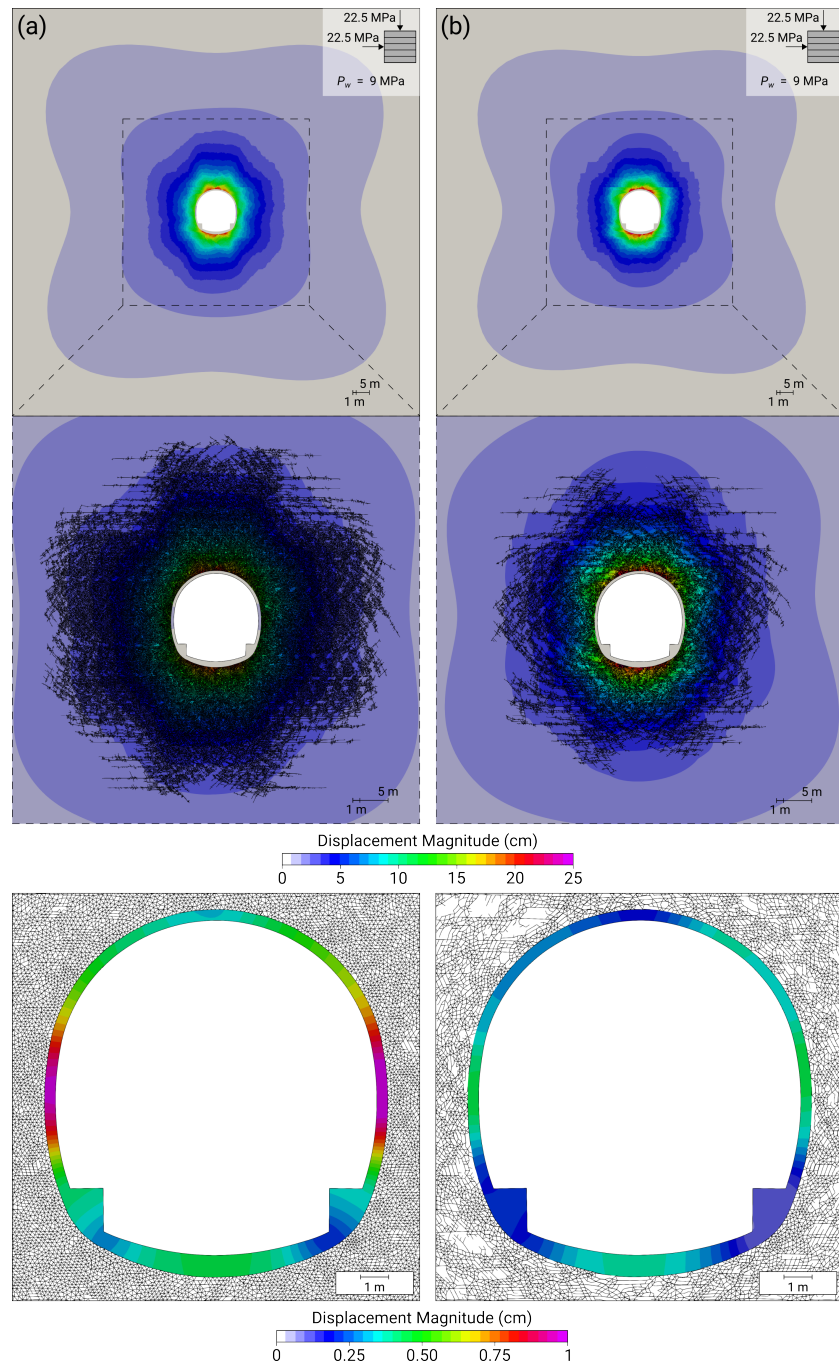


Fig. 5-10: Radial displacement contours and associated EDZ fracture pattern for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

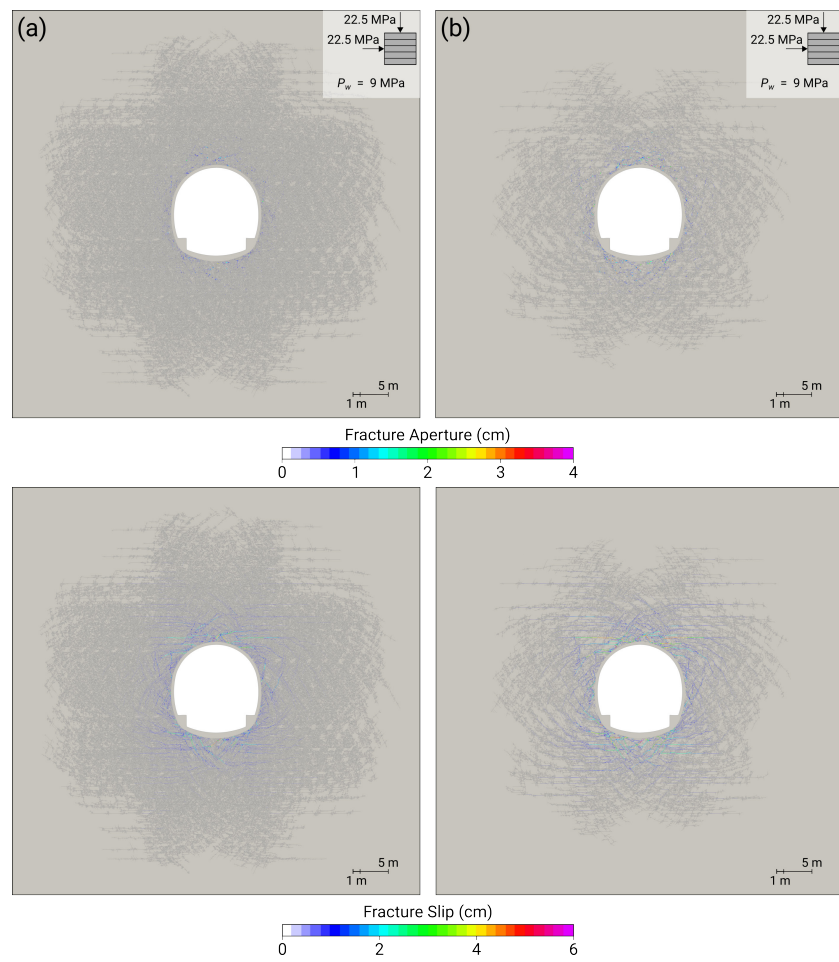


Fig. 5-11: Fracture aperture (top) and slip (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

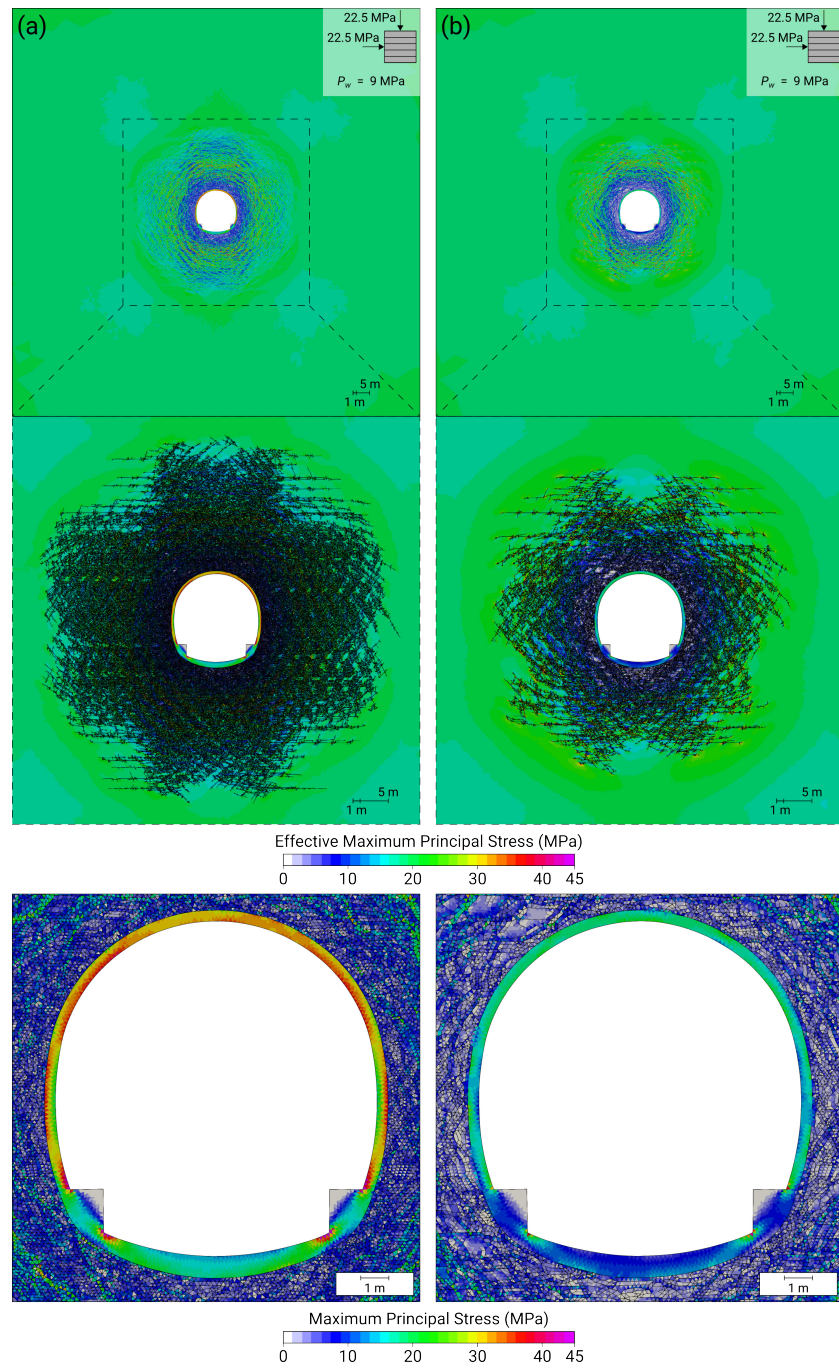


Fig. 5-12: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

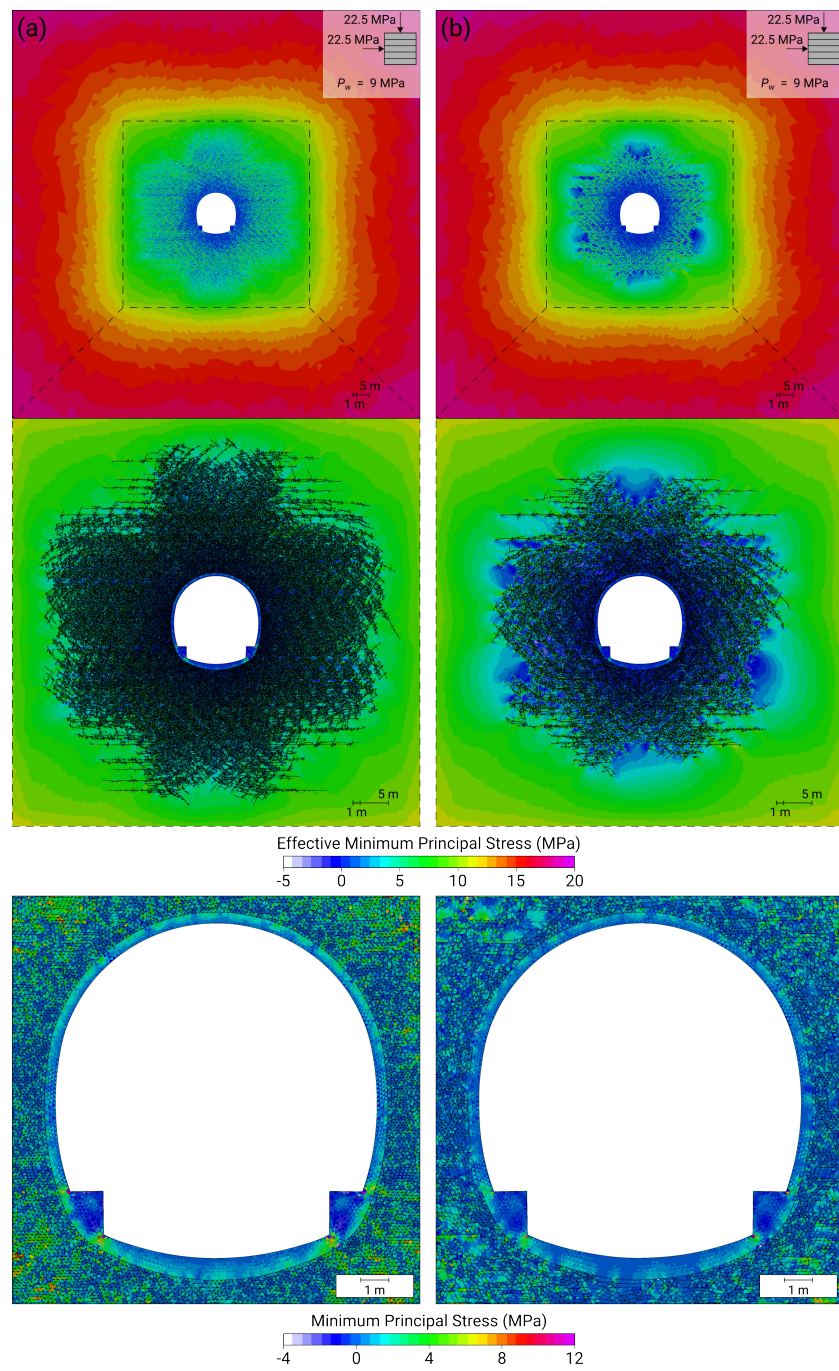


Fig. 5-13: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

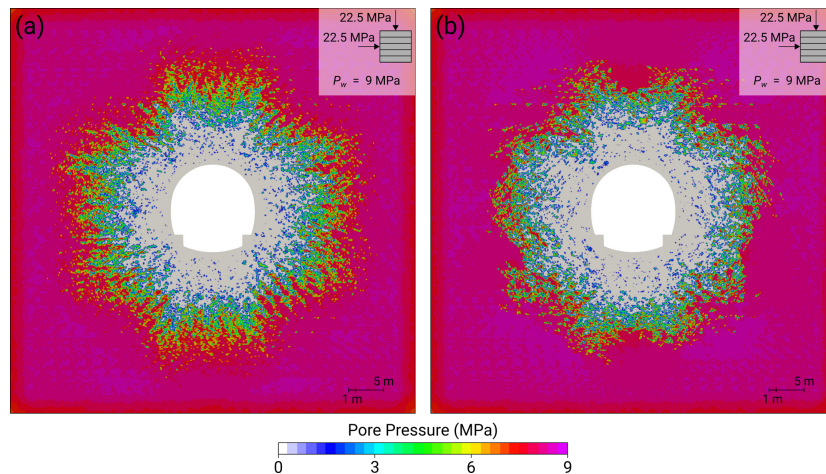


Fig. 5-14: Pore pressure contours for L/ILW cavern models with isotropic in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

5.3 Maximum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for L/ILW cavern models with maximum horizontal in-situ stress is shown in Fig. 5-15 (cases 5 and 6). The stiffness of the core and liner materials were reduced from its original values over 0.8 M time steps (from 0.3 M to 1.1 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0040$ for lower strength model and $\alpha_s = 0.00375$ for upper strength model) that resulted in approximately 20.5 cm (i.e., 3.3% of half the excavation span) of convergence at one of the four boundary monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.3 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 5-16, and as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. For the lower bound strength case, P₉₀ or the crown of the excavation reached the target convergence, while for the upper bound strength case, P₂₇₀ or the floor of the excavation reached the target convergence. Generally, the crown and floor of the excavation converged more than the sidewalls due to the anisotropic stiffness of the rock – where the rock was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.

The simulated displacement contours and EDZ for L/ILW cavern models with maximum horizontal in-situ stress is shown in Fig. 5-17. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result. After installing the stiff liner, there was 0.75-1.0 cm of liner displacement at the sidewalls, and 0.25-0.5 cm of liner displacement in the crown and floor for both rock strength cases. There was slightly more liner displacement simulated for the case of lower strength rock.

The extent of the EDZ for the lower bound strength simulation (Fig. 5-17a) were 13.5 m from the sidewalls and 19.6 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as an octagon elongated in the direction of the minor in-situ stress (i.e., crown/floor). The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersect and coalesce, a very dense fracture network emerges and discrete blocks that are approximately one

element size (i.e., $h = 0.15$ m) are formed near the excavation. The depth of this dense fracture network is approximately 6.8 m from the sidewall and 10.1 m from the crown/floor of the excavation.

The extent of the EDZ for the upper bound strength simulation (Fig. 5-17b) were 10.5 m from the sidewalls and 14.7 m from the crown/floor of the excavation. The overall shape of the EDZ can be described as an octagon elongated in the direction of the minor principal stress (i.e., crown/floor). The simulated EDZ contains bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and to a lesser extent, conjugate shear fractures in the sidewalls. Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 5-18. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement are bedding parallel fractures and intra-layer fractures approximately 4.0 m above/below the crown/floor. There was greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there was sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 5-19 and Fig. 5-20. Regions of greatest compressive stress were at the fracture tips of bedding parallel and intra-layer fracturing in the crown and floor of the excavation. This region correlated with high compressive stresses as a result of the stress redistribution of the far-field stresses around the excavation. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 5-21). In the liner, tangential stresses of up to 50 MPa and 45 MPa were simulated at the corners, the sidewall, and haunch of the liner for cases of lower and upper rock strength

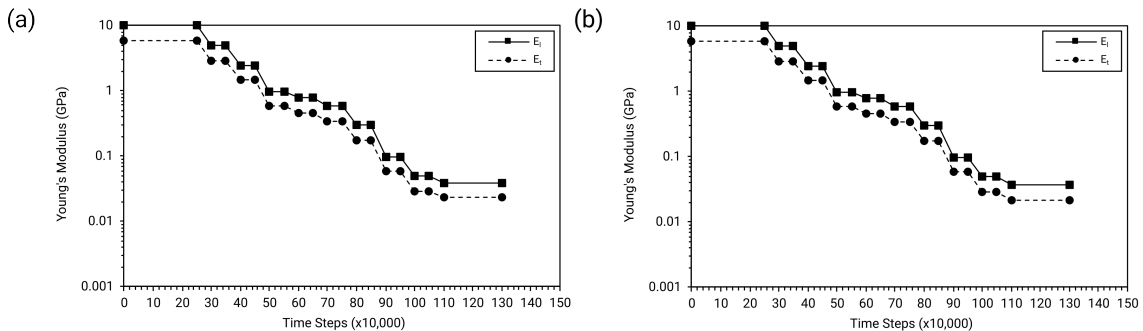


Fig. 5-15: The temporal softening scheme of the excavation core for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

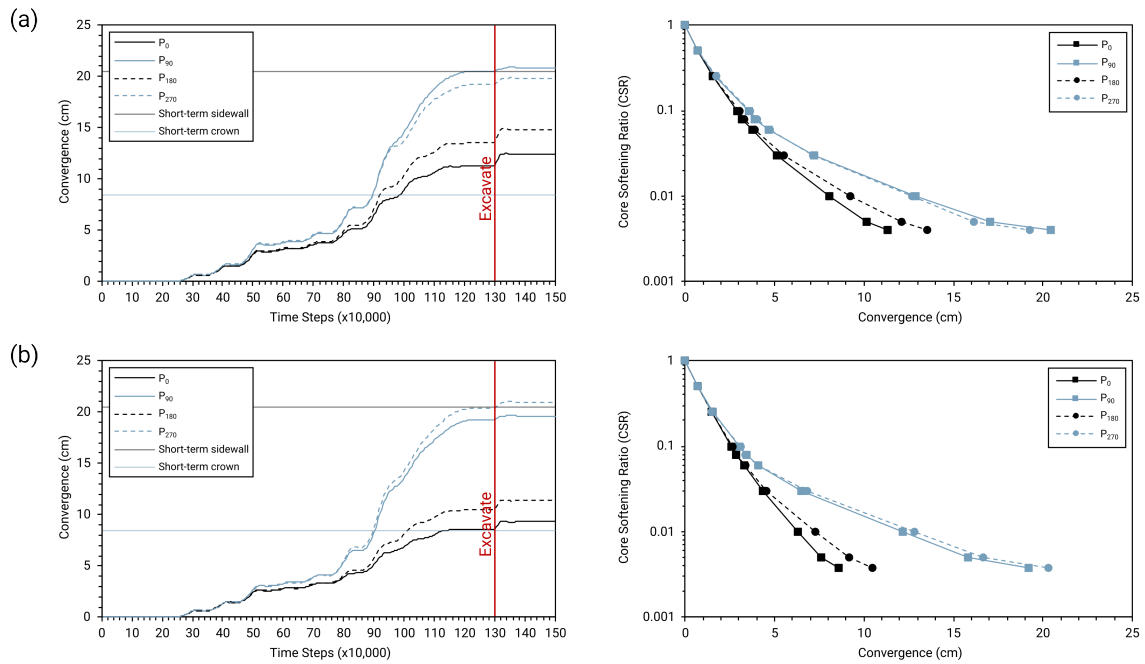


Fig. 5-16: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

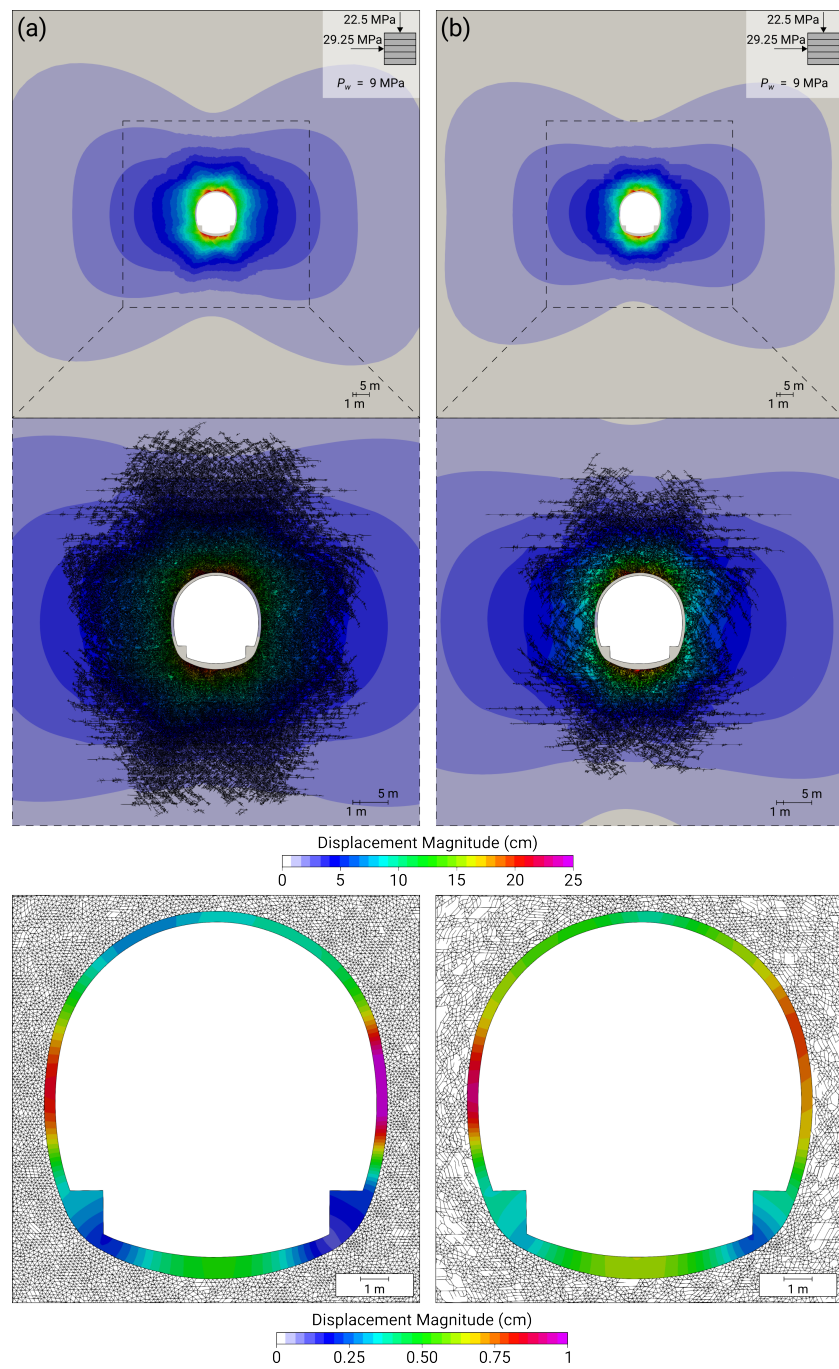


Fig. 5-17: Radial displacement contours and associated EDZ fracture pattern for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

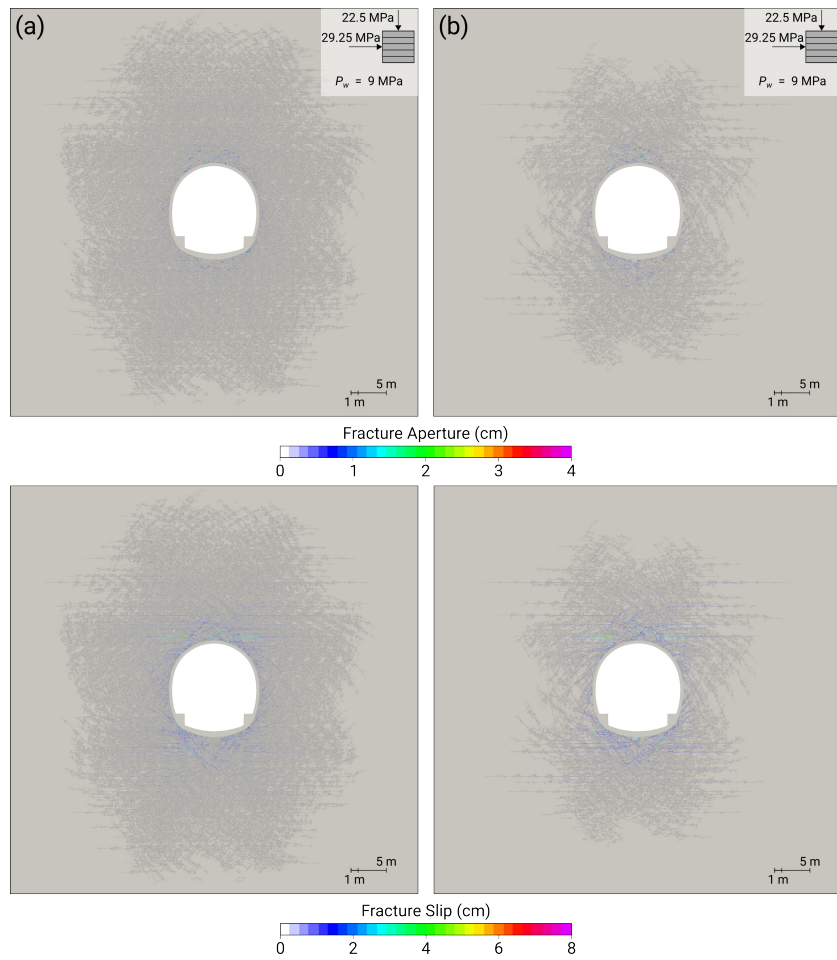


Fig. 5-18: Fracture aperture (top) and slip (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

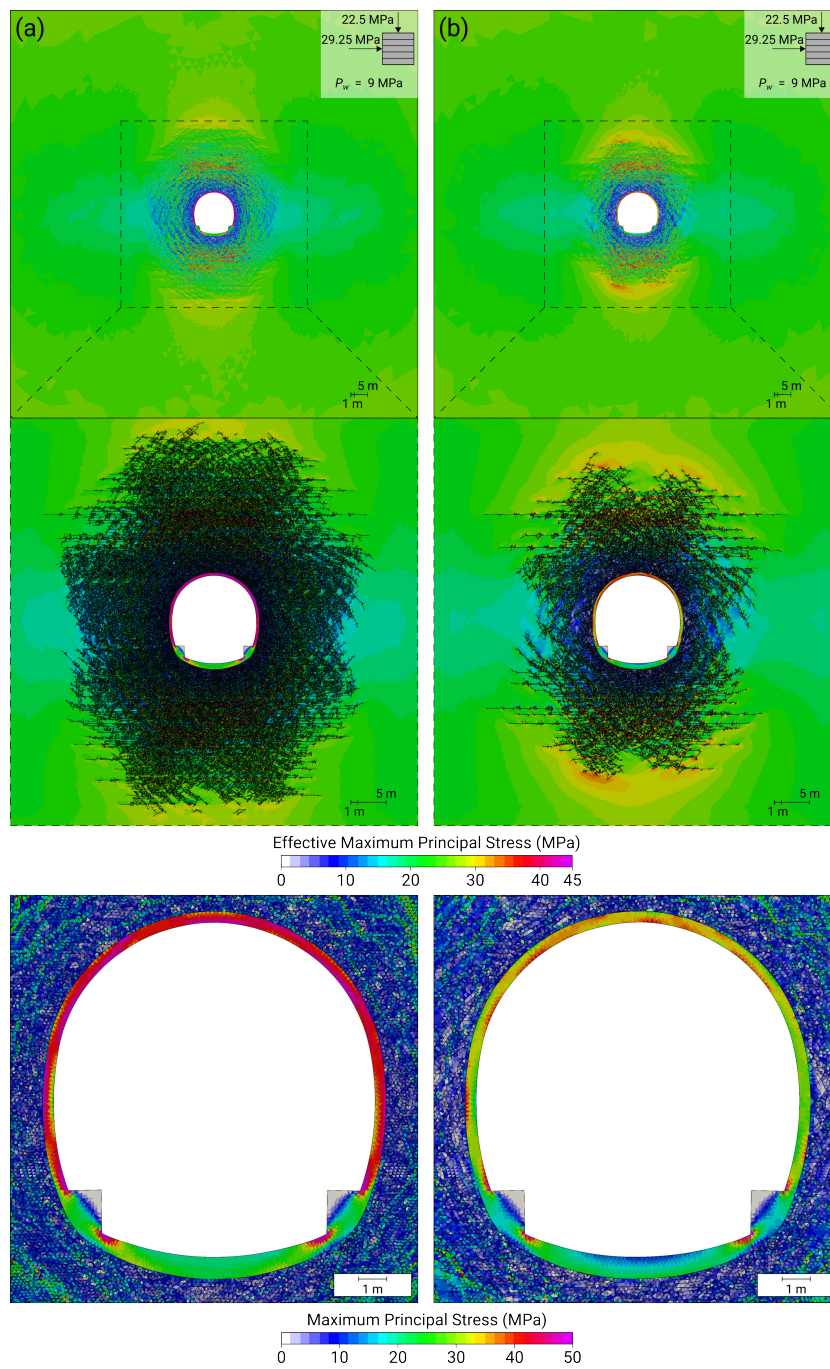


Fig. 5-19: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

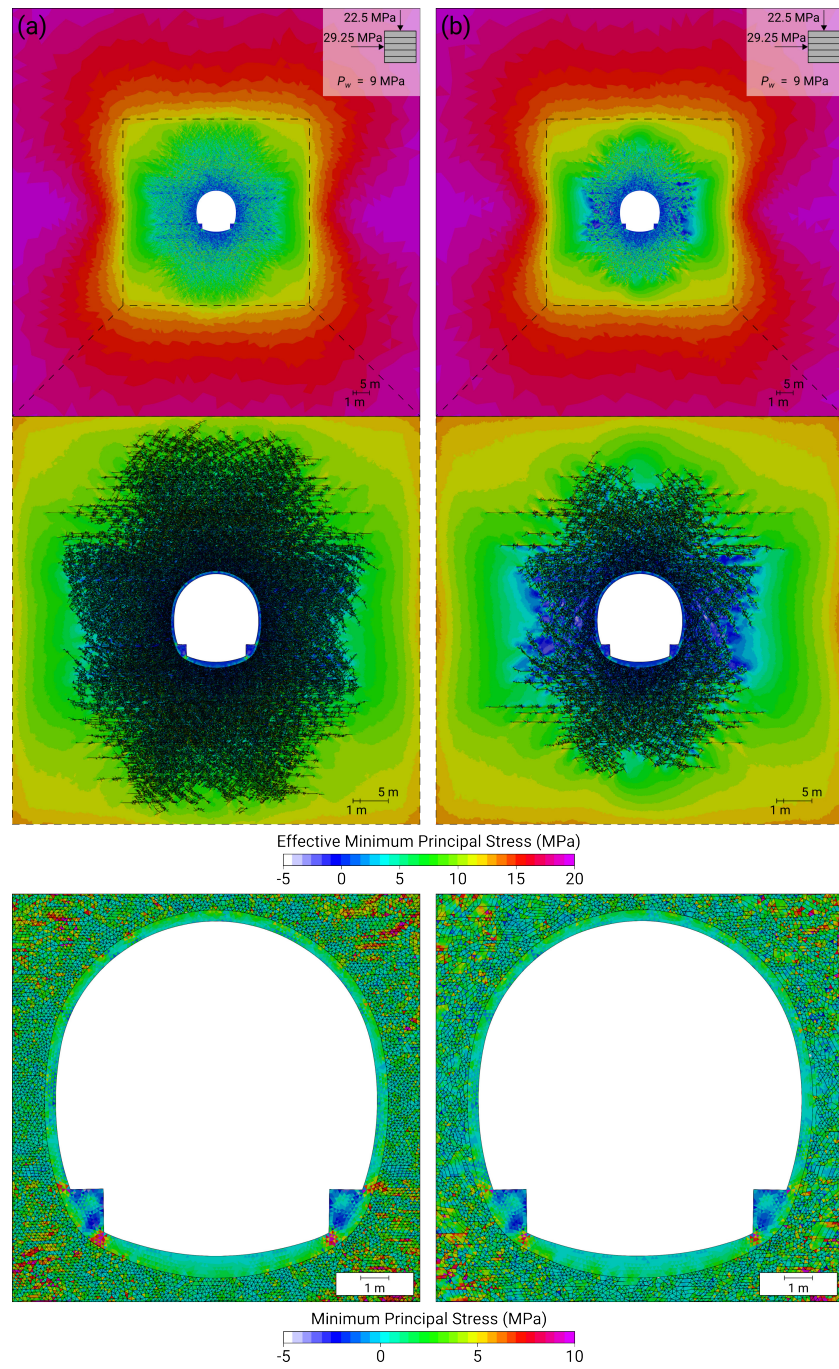


Fig. 5-20: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for L/ILW tunnel models with isotropic horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

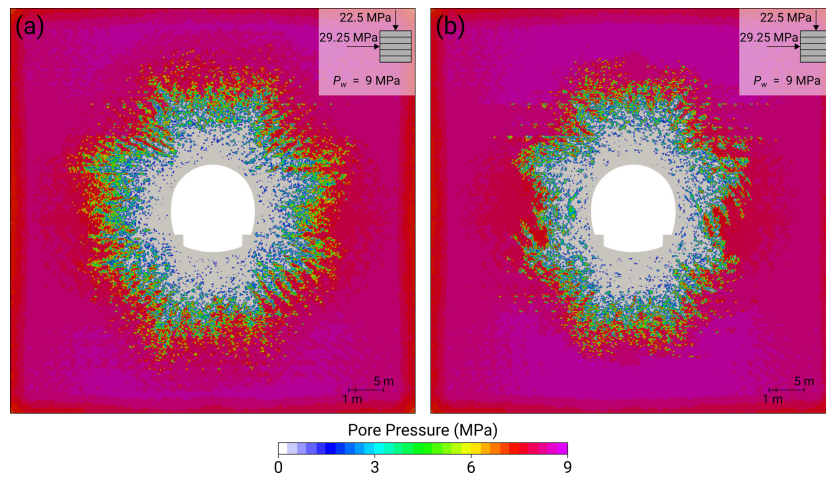


Fig. 5-21: Pore pressure contours for L/ILW cavern models with maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations.

6 Results: EDZ simulations for shaft

The EDZ was simulated for nine cases of the shaft, with varying horizontal in-situ stress, rock strength and core softening ratio. A summary of the simulated cases is shown in Tab. 6-1. For each model, the core softening ratio was adjusted such that the maximum deformation at one of the four monitoring points converged approximately 17.2 cm (3.3% of the excavation radius) before the liner was installed. The cross-section of the shaft was assumed to be parallel to the strike of the plane of isotropy. The material properties of the rock were assumed to be isotropic where the elastic properties in the direction parallel to bedding (i.e., E_l , ν_{ll}) and the maximum strength values (i.e., $f_{t,max}$, c_{max} , $G_{Ic,max}$, $G_{IIc,max}$) were used. In all models, it was assumed that one of the horizontal in-situ stress was constant and applied along the vertical model axis (i.e., $\sigma_{yy} = 22.5$ MPa), while the other in-plane horizontal stress was varied. For all models, an initial pore pressure of 9 MPa was applied to the rock mass, with the liner being prescribed zero pore pressure. In the following sub-sections, the results are presented in the form of ground reaction curves, evolution of displacement of select points located on the excavation boundary, fracture patterns, contours of displacement, contours of effective stress (in the rock mass) and total stress (in the lining), and contours of fracture aperture and slip.

Tab. 6-1: Summary of shaft simulations performed in this study

Case	Total minimum horizontal stress, σ_{xx} [MPa]	Total maximum horizontal stress, σ_{yy} [MPa]	Rock strength properties	Core softening scheme	Core softening ratio, α_s
1	17.5 (σ_{xx})	22.5 (σ_{yy})	lower	isotropic	0.00040
2	17.5 (σ_{xx})	22.5 (σ_{yy})	upper	isotropic	0.00010
3	22.5	22.5	lower	isotropic	0.00045
4	22.5	22.5	upper	isotropic	0.00010
5	22.5 (σ_{yy})	29.25 (σ_{xx})	lower	isotropic	0.00095
6	22.5 (σ_{yy})	29.25 (σ_{xx})	upper	isotropic	0.00050
7	17.5 (σ_{xx})	22.5 (σ_{yy})	lower	isotropic	0.004
8	22.5	22.5	lower	isotropic	0.004
9	22.5 (σ_{yy})	29.25 (σ_{xx})	lower	isotropic	0.007

6.1 Minimum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for shaft models with minimum horizontal in-situ stress (Tab. 6-1, cases 1 and 2) is shown in Fig. 6-1. The stiffness of the core and liner materials were reduced from its original values over 0.7 M time steps (from 0.3 M to 1.0 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.0004$ for lower strength model and $\alpha_s = 0.0001$ for upper strength model) that resulted in approximately 17.2 cm (i.e., 3.3% of the shaft radius) of convergence at one of the four monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.35 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 6-2, as expected, the walls of the excavation converged as the Young's modulus of the core and liner were reduced. In both rock strength cases, monitoring points P_0 and P_{180} reached approximately 17.2 cm of convergence and generally, the excavation walls along the axis of σ_h converged more than the axis of σ_H due to the anisotropic in-situ stress field. Comparing the $P_{90/270}$ displacement between the two strength cases, there was approximately 4-6 cm more displacement in the lower strength case and hence greater displacement anisotropy for the upper strength case. This observation correlated with reduced fracturing in the $P_{90/270}$ directions of the shaft as the strength of the rock was increased.

The simulated displacement contours and EDZ for shaft models with minimum horizontal in-situ stress is shown in Fig. 6-3. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). After installing the stiff liner, there was negligible additional convergence/divergence simulated. In the case of upper rock strength, liner divergence was simulated at points $P_{90/270}$ of the liner. This phenomenon was a result of the liner supporting the fractured rock mass in the direction of σ_h and there being relatively less fracturing in the direction of σ_H .

The extent of the EDZ for the lower bound strength simulation (Fig. 6-3a) was 21.1 m in the direction of σ_h and 13.0 m in the direction of σ_H of the excavation. The overall shape of the EDZ can be described as eye-shaped that was elongated in the direction of σ_h . The simulated EDZ contained conjugate shear fractures around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) were formed approximately a depth of 7 m from $P_{0/180}$ and 10 m from $P_{90/270}$.

The extent of the EDZ for the upper bound strength simulation (Fig. 6-3b) was 15.6 m in the direction of σ_h and 5.7 m in the direction of σ_H of the excavation. The overall shape of the EDZ can be described as a lemniscate. The simulated EDZ contains conjugate shear fractures around the excavation, with preferential fracturing in the regions in the direction of σ_h . Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 6-4. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement were conjugate shear fractures around the excavation, approximately 4-8 m from the excavation boundary.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 6-5 and Fig. 6-6. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures in the σ_h direction of the shaft. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 6-7). In the liner, tangential stresses up to 4 MPa and 5 MPa were simulated in the direction of σ_H for the case of lower and upper strength.

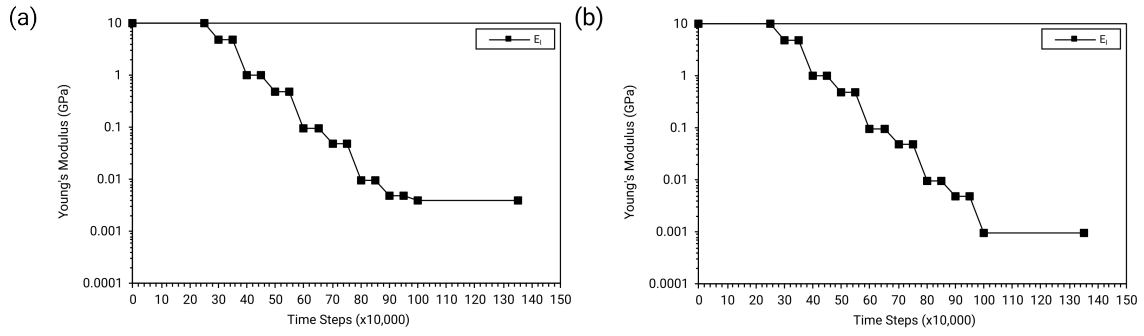


Fig. 6-1: The temporal softening scheme of the excavation core for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

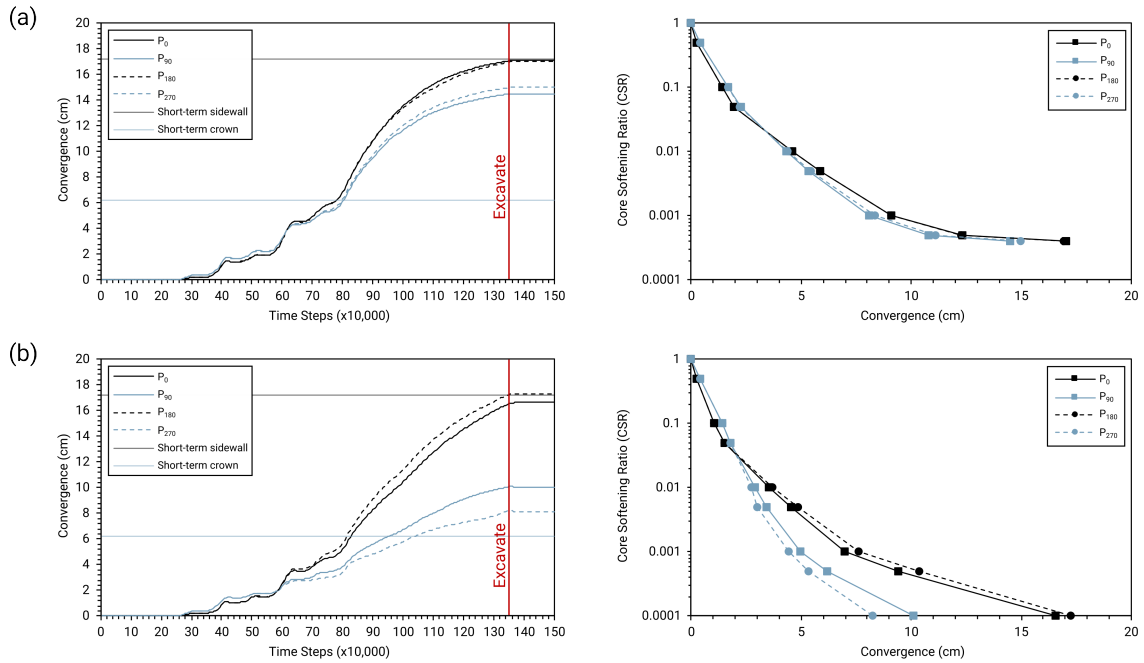


Fig. 6-2: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

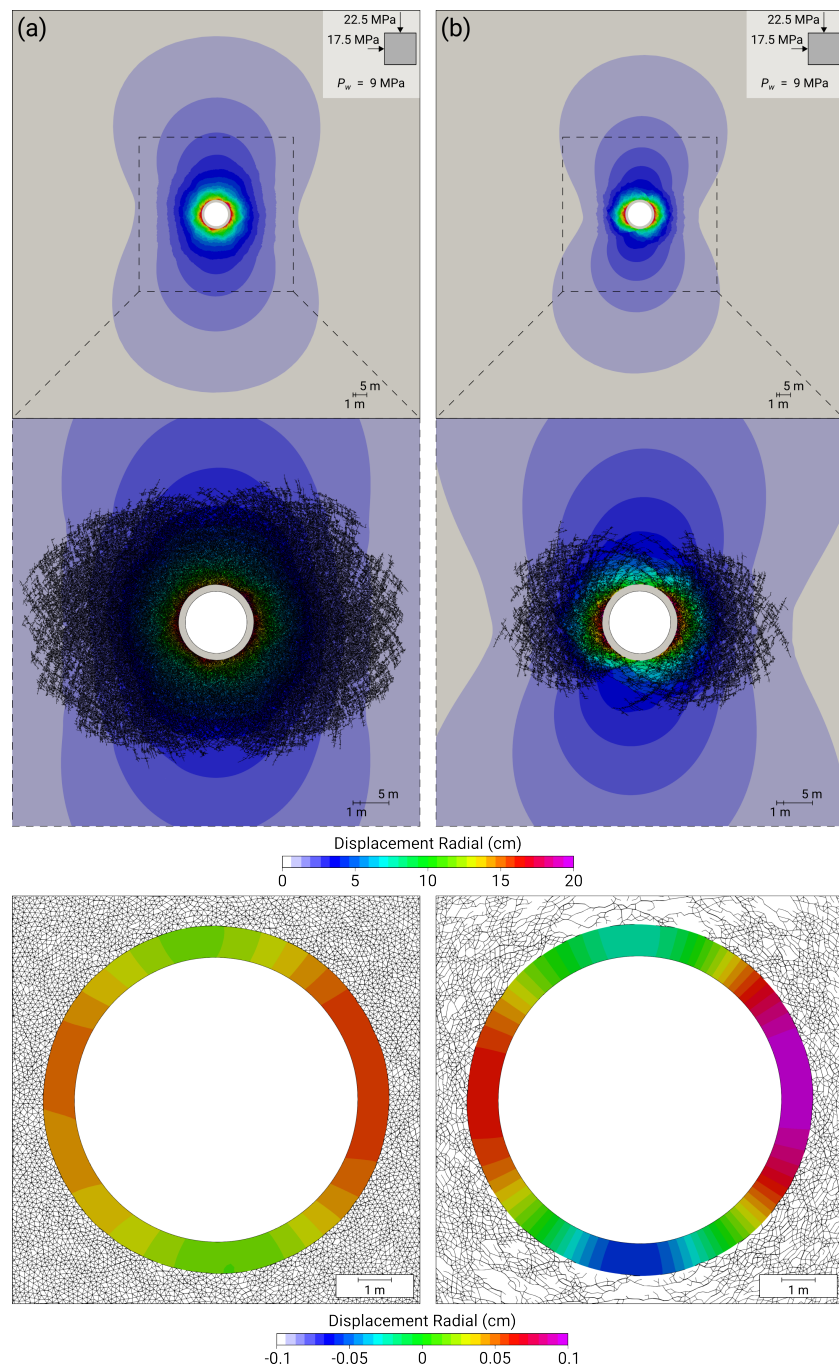


Fig. 6-3: Radial displacement contours and associated EDZ fracture pattern for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

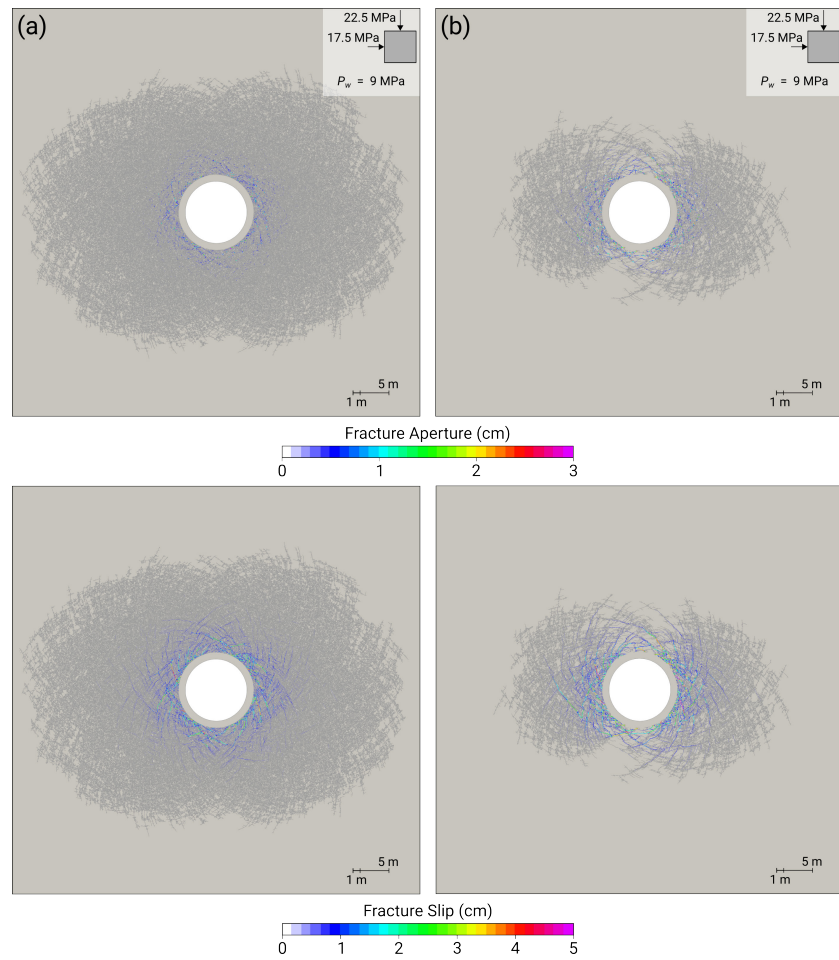


Fig. 6-4: Fracture aperture (top) and slip (bottom) contours for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

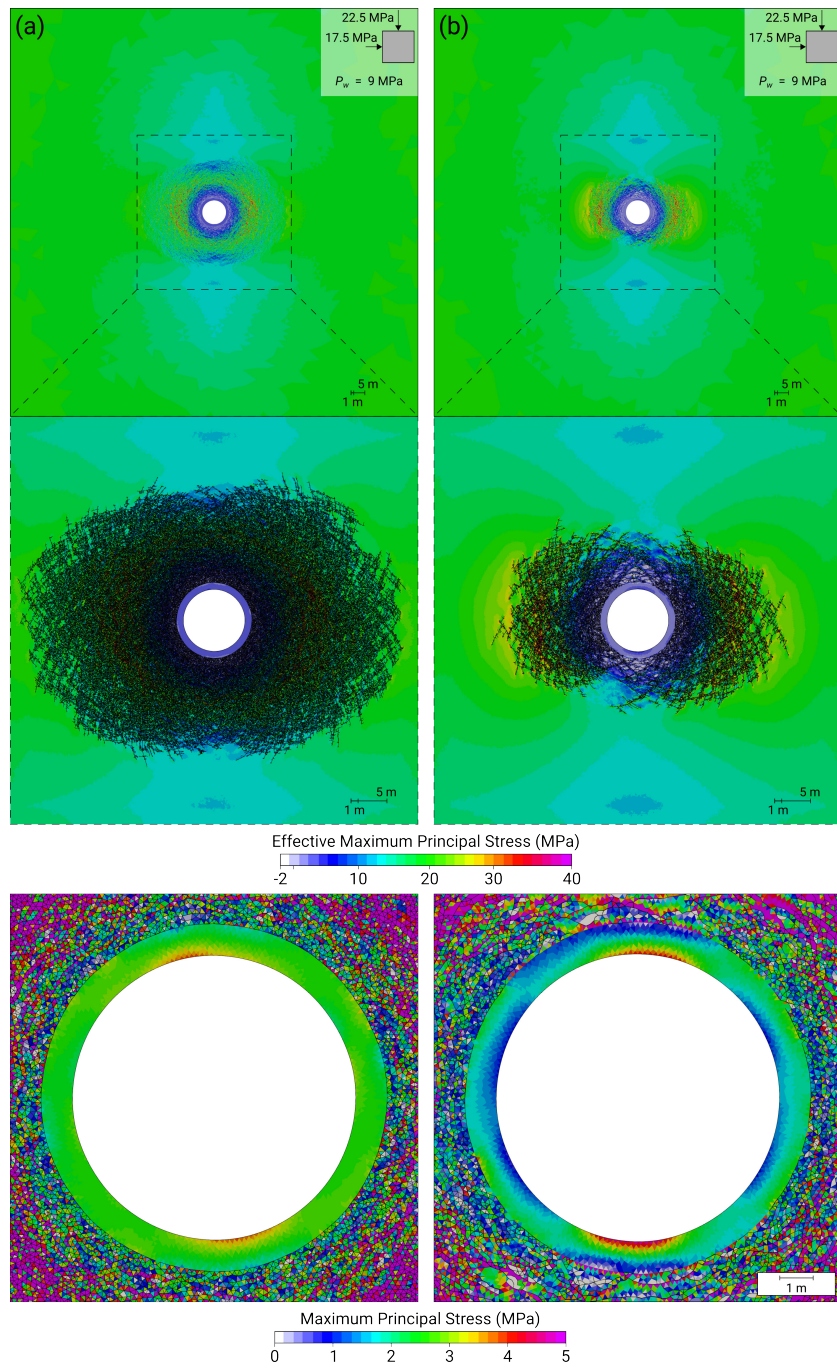


Fig. 6-5: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective maximum principal stress: maximum principal stress subtracted with the pore pressure. Positive stress is compression.

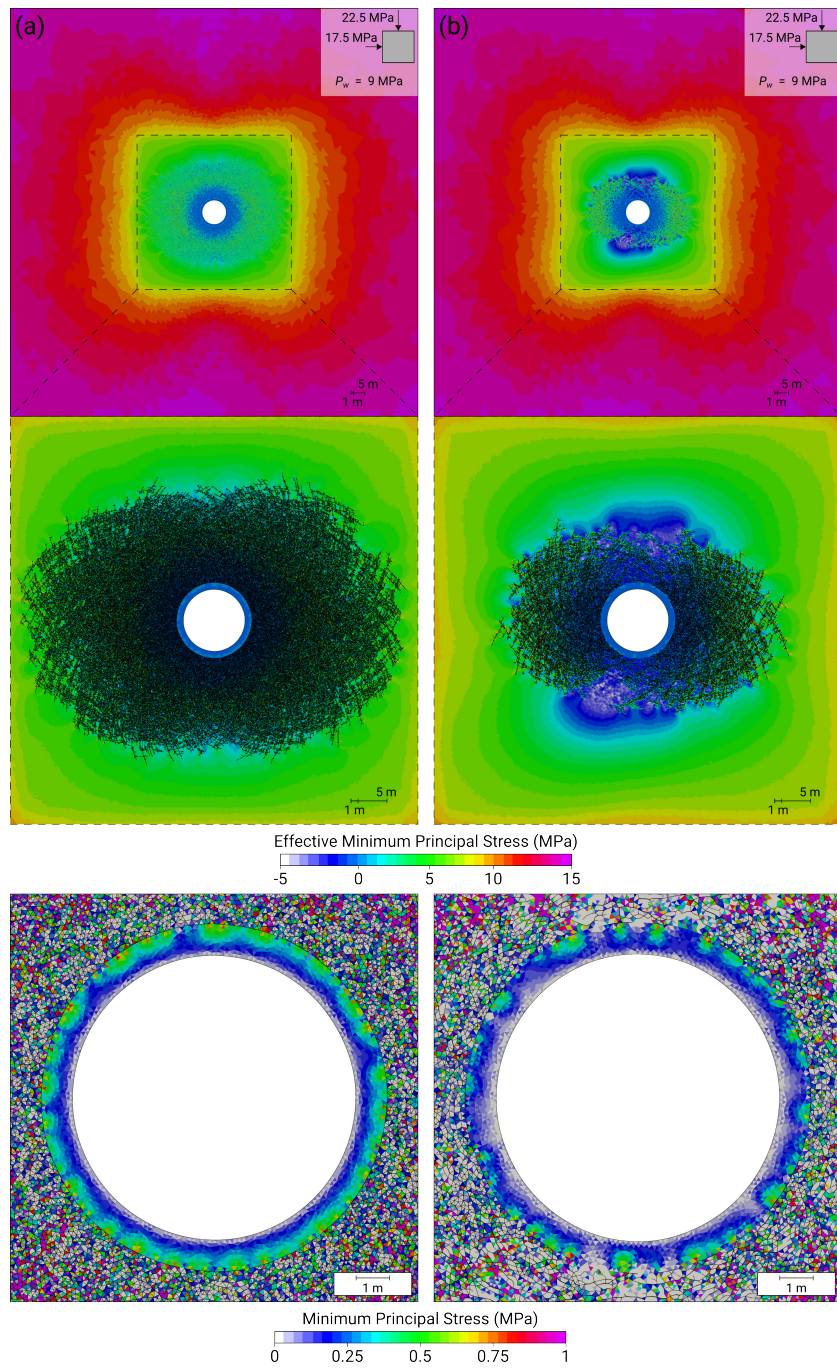


Fig. 6-6: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Effective minimum principal stress: minimum principal stress subtracted with the pore pressure. Positive stress is compression.

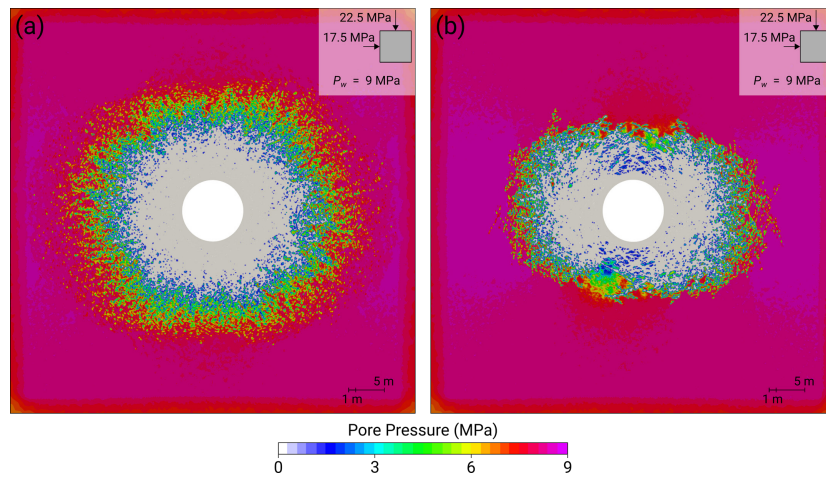


Fig. 6-7: Pore pressure contours for shaft models with minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

6.2 Isotropic in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for shaft models with isotropic in-situ stress is shown in Fig. 6-8 (cases 3 and 4). The stiffness of the core and liner materials were reduced from its original values over 0.7 M time steps (from 0.3 M to 1.0 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.00045$ for lower strength model and $\alpha_s = 0.0001$ for upper strength model) that resulted in approximately 17.2 cm (i.e., 3.3% of the shaft radius) of convergence at one of the four monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.35 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 6-9, as expected, the walls of the excavation converged as the Young's modulus of the core and liner were reduced. In both rock strength cases, displacement at the four monitoring points were fairly isotropic and all reached approximately 17.2 cm of convergence.

The simulated displacement contours and EDZ for shaft models with isotropic in-situ stress is shown in Fig. 6-10. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). After installing the stiff liner, there was negligible additional convergence simulated (up to 0.08 cm).

The extent of the EDZ for the lower bound strength simulation (Fig. 6-10a) were 16.0 m from the four monitoring points at the excavation boundary. The overall shape of the EDZ can be described as a circle. The simulated EDZ contains conjugate shear fractures all around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) was formed at a depth of 8 m from the excavation boundary.

The extent of the EDZ for the upper bound strength simulation (Fig. 6-10b) were 11.5 m from the four monitoring points at the excavation boundary. The overall shape of the EDZ can be described as a circle. The simulated EDZ contains conjugate shear fractures around the excavation. Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case. As expected, the extent of fracturing was greater for the case of lower rock strength than upper rock strength.

The aperture and slip of the fractures for the simulations are shown in Fig. 6-11. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement were conjugate shear fractures approximately 5 m into the rock mass all around the excavation boundary. There was greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there was sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 6-12 and Fig. 6-13. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures at the extents of the EDZ. There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 6-14). In the liner, tangential stresses of up to 4.5 MPa were simulated in the principal directions for the case of lower rock strength. For the case of upper rock strength, tangential stresses up to 4 MPa were simulated, but stresses in the liner were fairly uniform at 2.5 MPa.

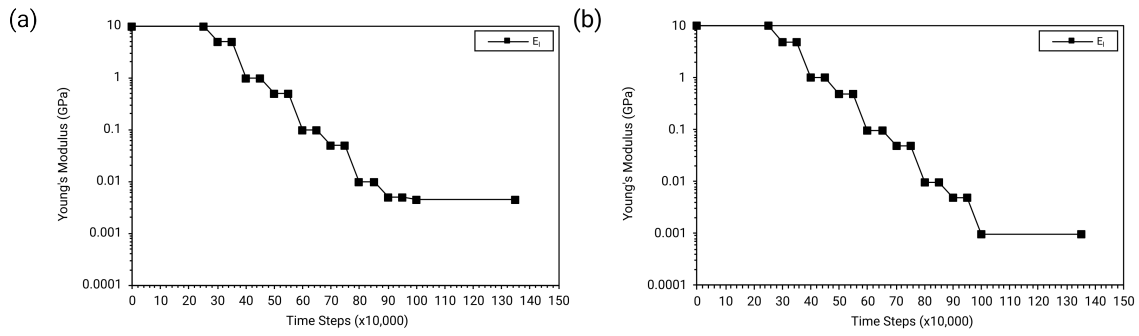


Fig. 6-8: The temporal softening scheme of the excavation core for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

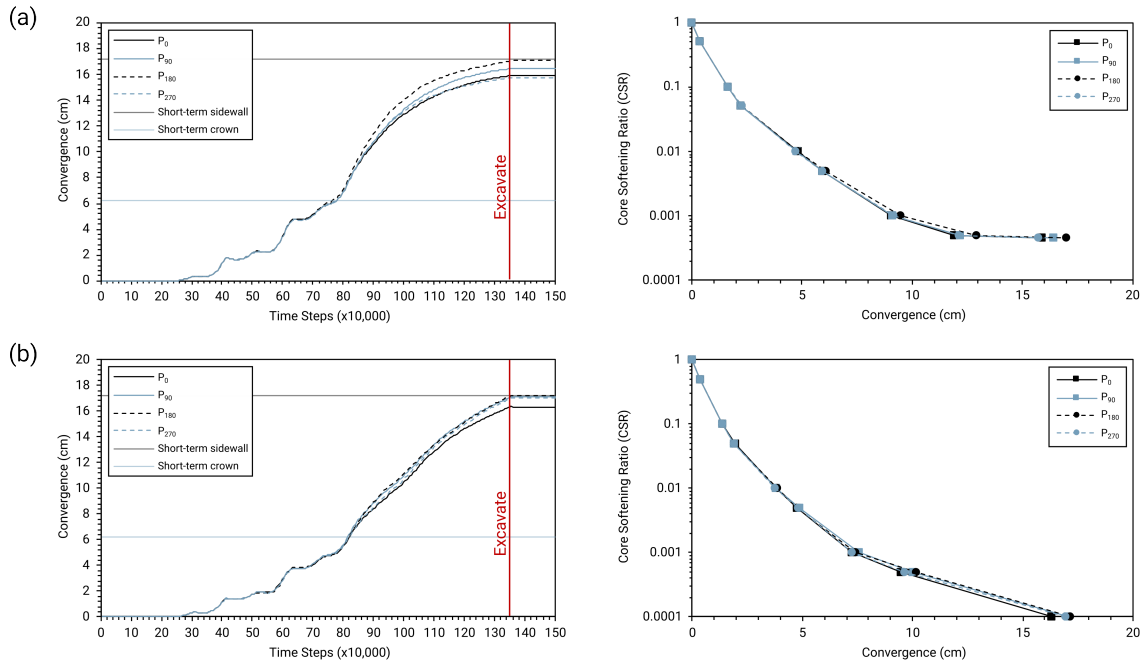


Fig. 6-9: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

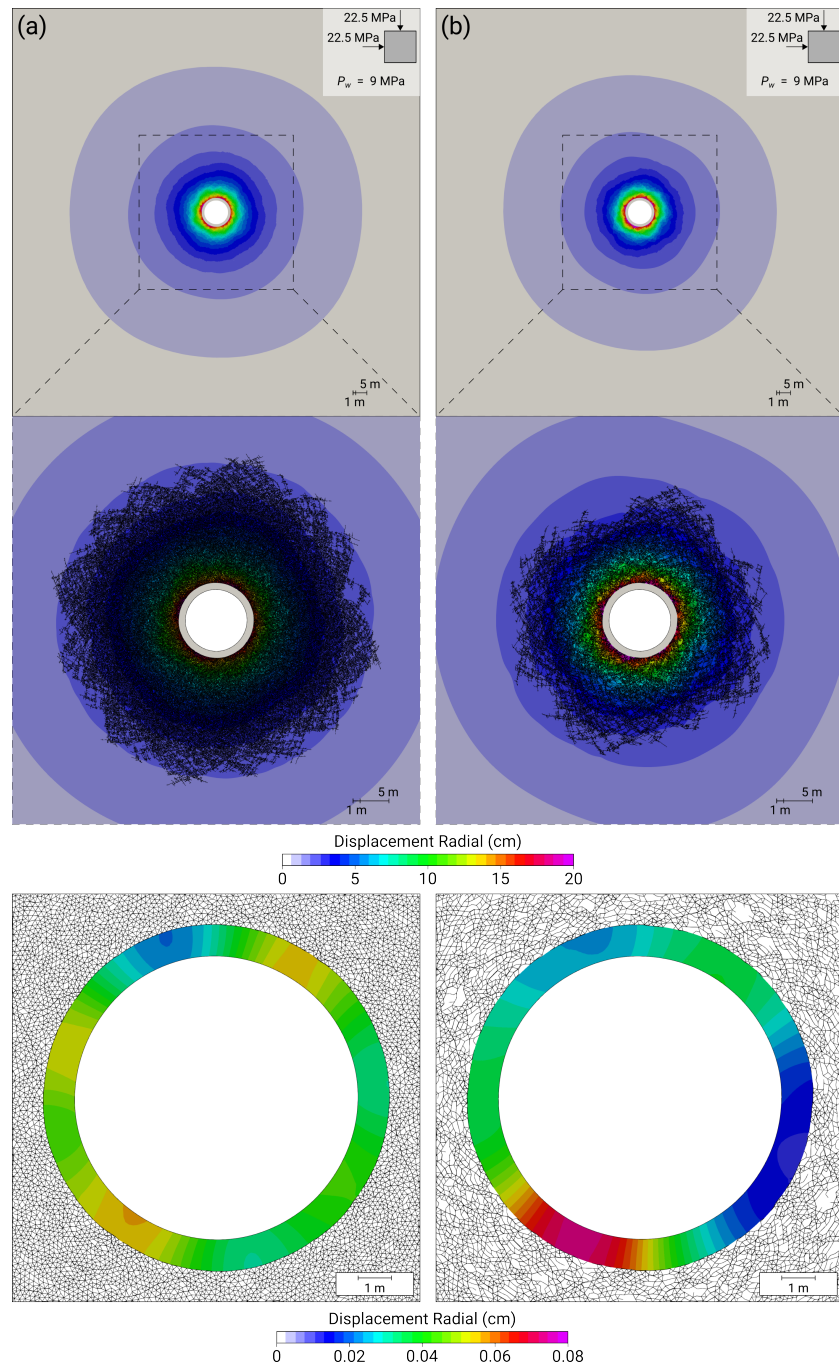


Fig. 6-10: Radial displacement contours and associated EDZ fracture pattern for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

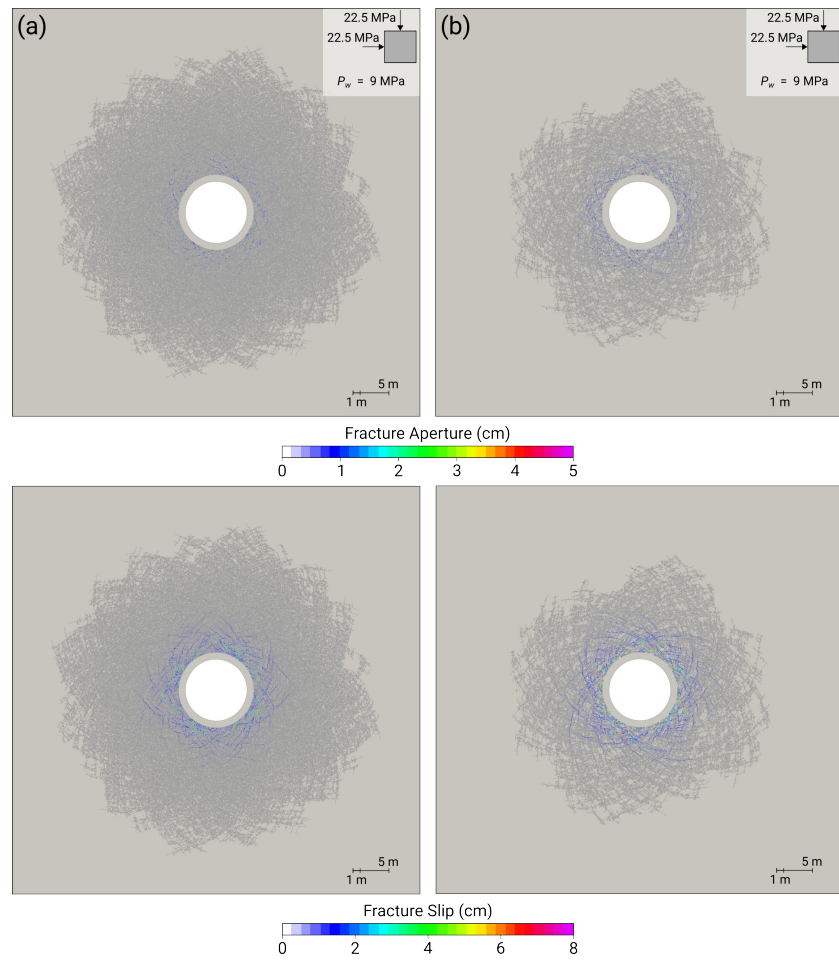


Fig. 6-11: Fracture aperture (top) and slip (bottom) contours for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

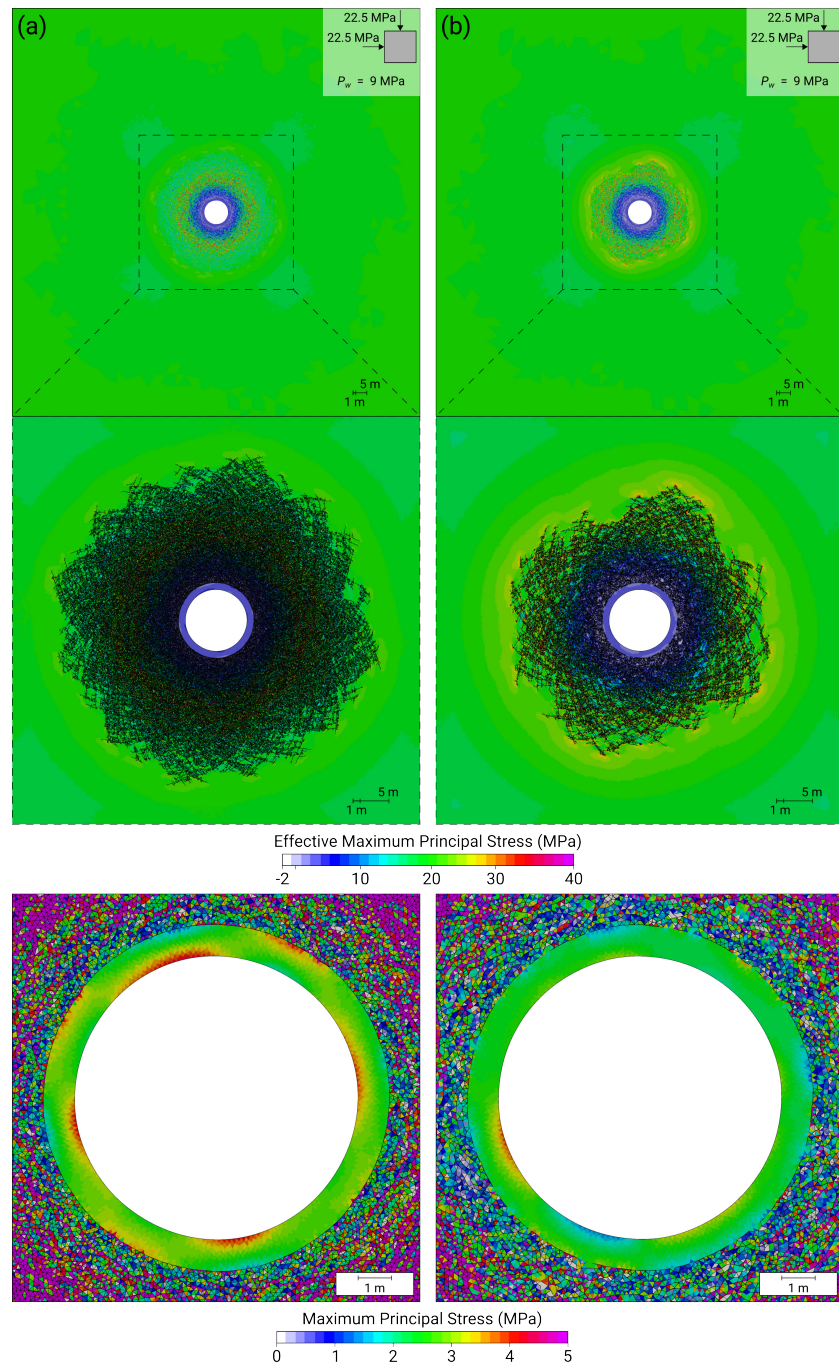


Fig. 6-12: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Positive stress is compression.

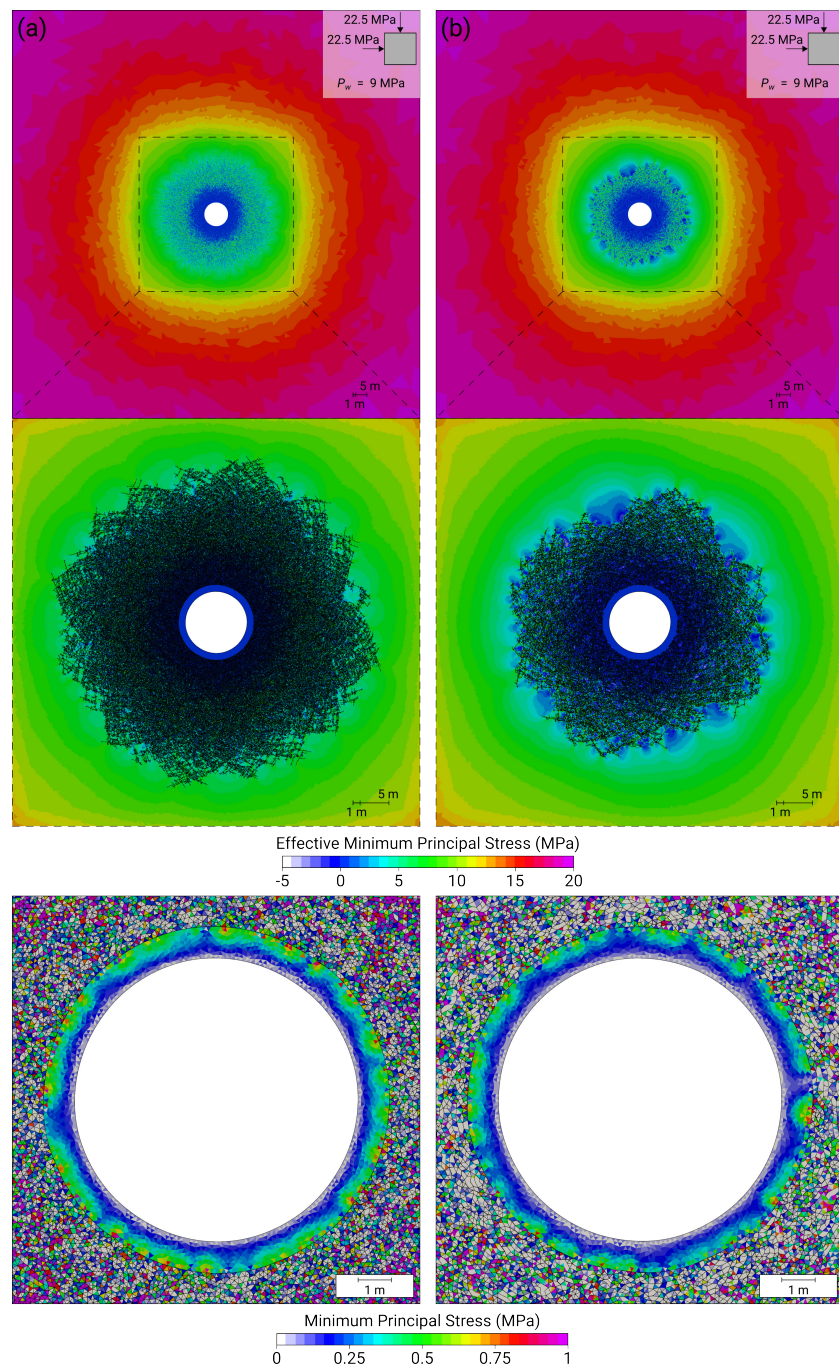


Fig. 6-13: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for shaft models with isotropic horizontal in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Positive stress is compression.

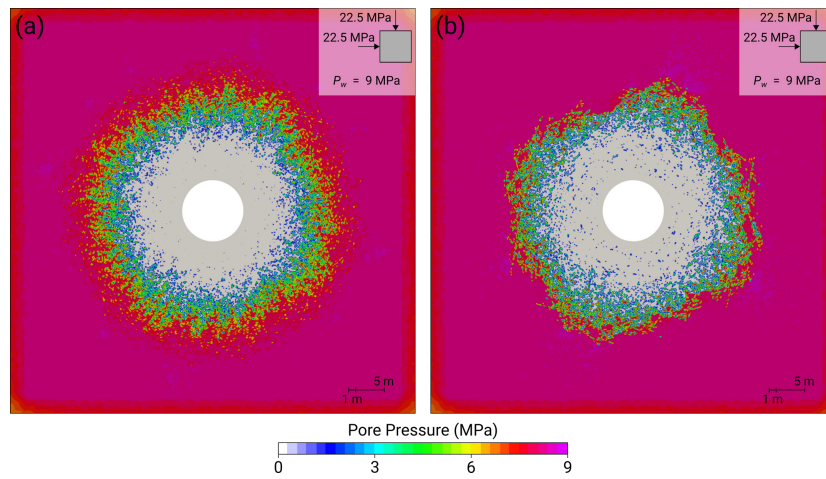


Fig. 6-14: Pore pressure contours for shaft models with isotropic in-situ stress ($\sigma_h = 22.5$ MPa) for (a) lower bound and (b) upper bound strength calibrations

6.3 Maximum horizontal in-situ stress: sensitivity to rock strength

The Young's modulus reduction scheme for shaft models with maximum horizontal in-situ stress is shown in Fig. 6-15 (cases 5 and 6). The stiffness of the core and liner materials were reduced from its original values over 0.5-0.6 M time steps (from 0.3 M to 0.8-0.9 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.00095$ for lower strength model and $\alpha_s = 0.0005$ for upper strength model) that resulted in approximately 17.2 cm (i.e., 3.3% of the shaft radius) of convergence at one of the four boundary monitoring points. The stronger strength calibration required a slightly reduced core softening ratio to achieve the same convergence as there was significantly less fracturing and hence displacement near the excavation boundary. The core was subsequently excavated at 1.35 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 6-16, as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. In both rock strength cases, monitoring points P_{90/270} reached approximately 17.2 cm of convergence and generally, the excavation walls along the axis of σ_h converged more than the axis of σ_H due to the anisotropic in-situ stress field.

The simulated displacement contours and EDZ for shaft models with maximum horizontal in-situ stress is shown in Fig. 6-17. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). After installing the stiff liner, there was negligible additional convergence simulated. Generally, there was greater convergence at monitoring points P_{90/270} as there was more fracturing in these regions of the excavation and hence greater rock mass displacement that the liner was supporting.

The extent of the EDZ for the lower bound strength simulation (Fig. 6-17a) were 16.2 m in the direction of σ_h and 10.0 m in the direction of σ_H of the excavation. The overall shape of the EDZ could be described as eye-shaped that was elongated in the direction of σ_h . The simulated EDZ contains conjugate shear fractures around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) were formed approximately a depth of 6.7 m from P_{0/180} and 9.4 m from P_{90/270}.

The extent of the EDZ for the upper bound strength simulation (Fig. 6-17b) were 12.9 m in the direction of σ_h and 8.0 m in the direction of σ_H of the excavation. The overall shape of the EDZ could be described as eye-shaped that was elongated in the direction of σ_h . The simulated EDZ contains conjugate shear fractures around the excavation, with preferential fracturing in the regions in the direction of σ_h . Overall, the simulated fracture network for the upper bound strength case was significantly less dense compared to the lower strength case.

The aperture and slip of the fractures for the simulations are shown in Fig. 6-18. As observed in the plot, the region near the excavation boundary has the greatest aperture and slip. Regions of significant shear displacement were conjugate shear fractures, particularly in the direction of σ_h , approximately 4.5-5.5 m into the rock mass. In the direction of σ_H , there was significant shearing approximately 3 m into the rock mass. There was greater simulated slip and aperture for the case of upper bound rock strength as due to the significantly less dense EDZ, there was sparse interlocking within the fractured rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 6-19 and Fig. 6-20. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures, particularly in the direction of σ_h . There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 6-21). In the liner, tangential stresses of up to 7 MPa and 5 MPa were simulated in the direction of σ_H .

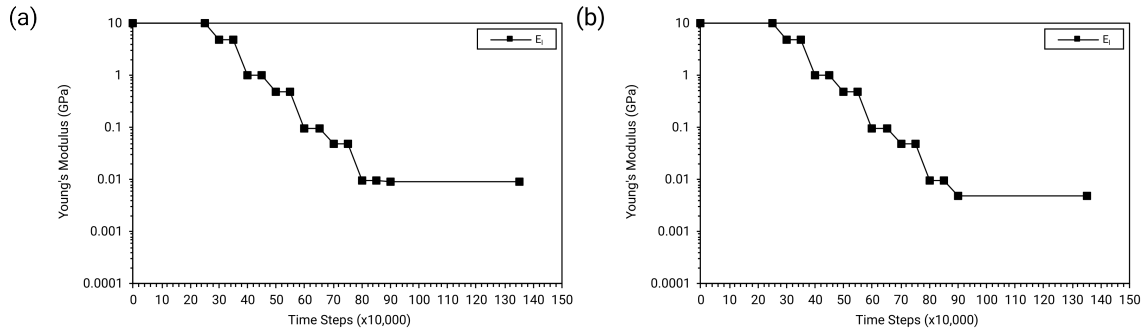


Fig. 6-15: The temporal softening scheme of the excavation core for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

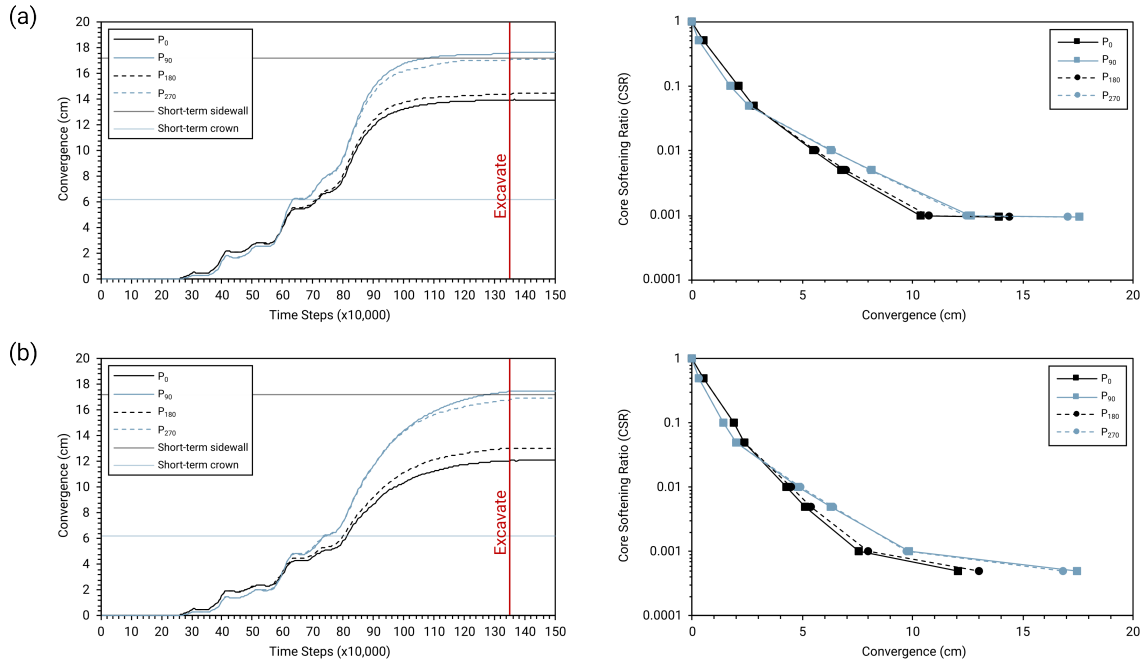


Fig. 6-16: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

For reference, the provided short-term sidewall and crown convergences by Nagra are plotted as horizontal lines. Positive convergence is towards the centre of the excavation.

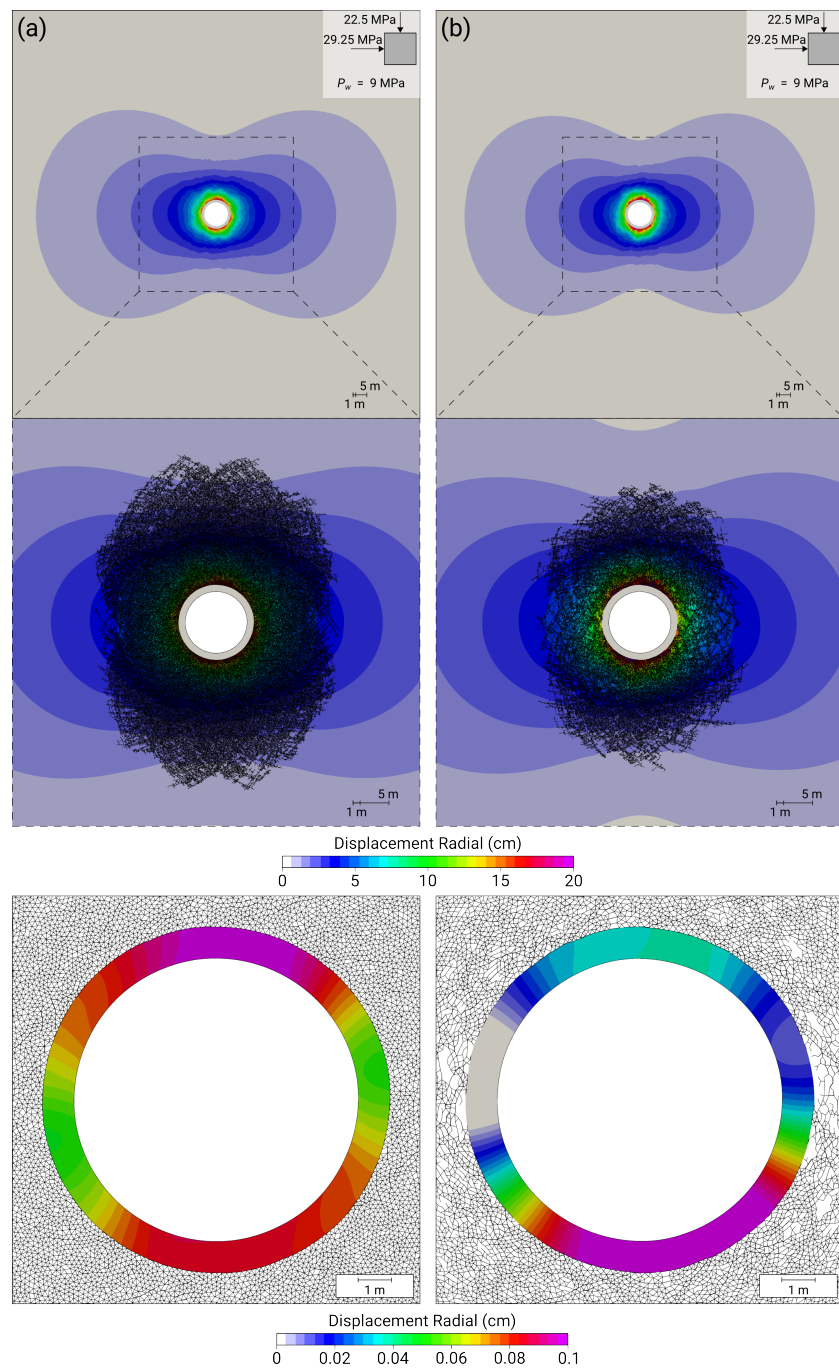


Fig. 6-17: Radial displacement contours and associated EDZ fracture pattern for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25 \text{ MPa}$) for (a) lower bound and (b) upper bound strength calibrations

The liner displacement shown is the displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

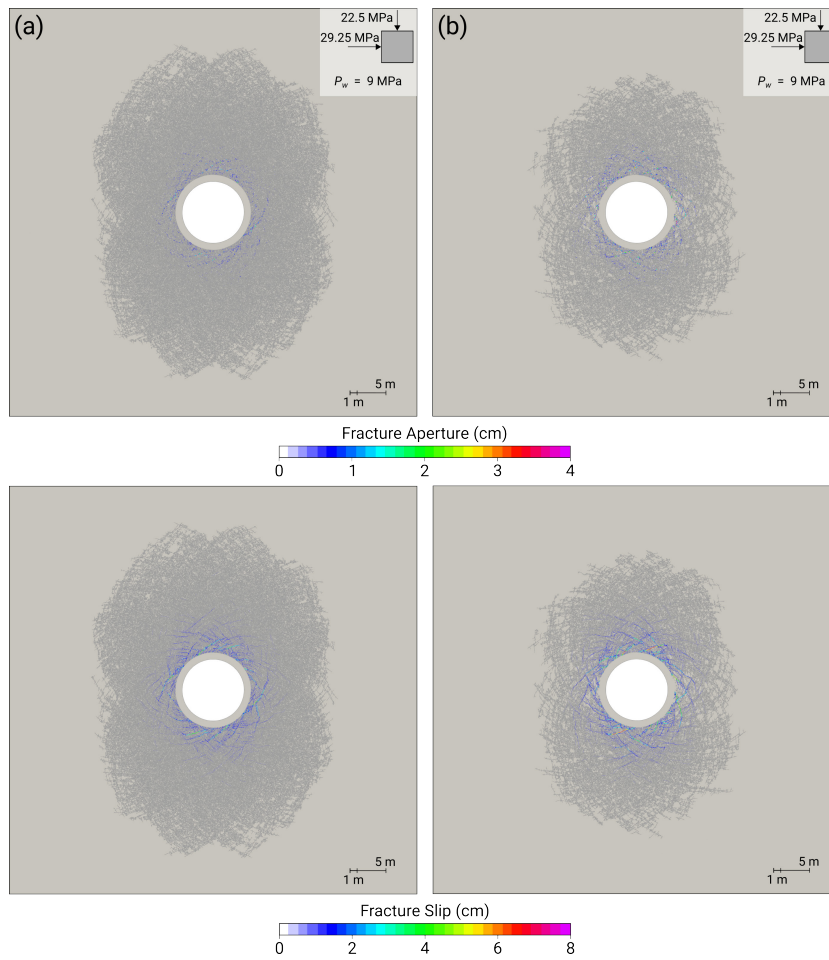


Fig. 6-18: Fracture aperture (top) and slip (bottom) contours for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

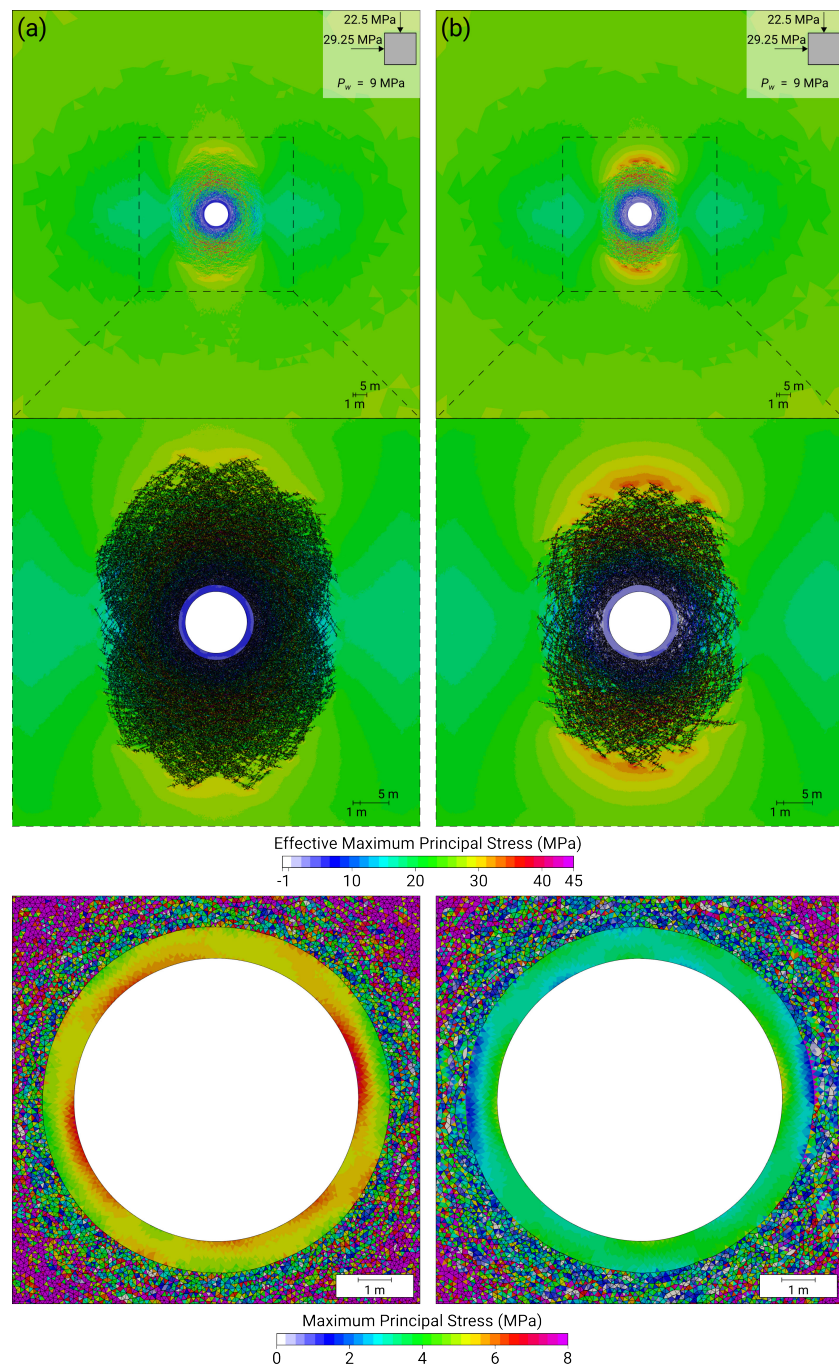


Fig. 6-19: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Positive stress is compression.

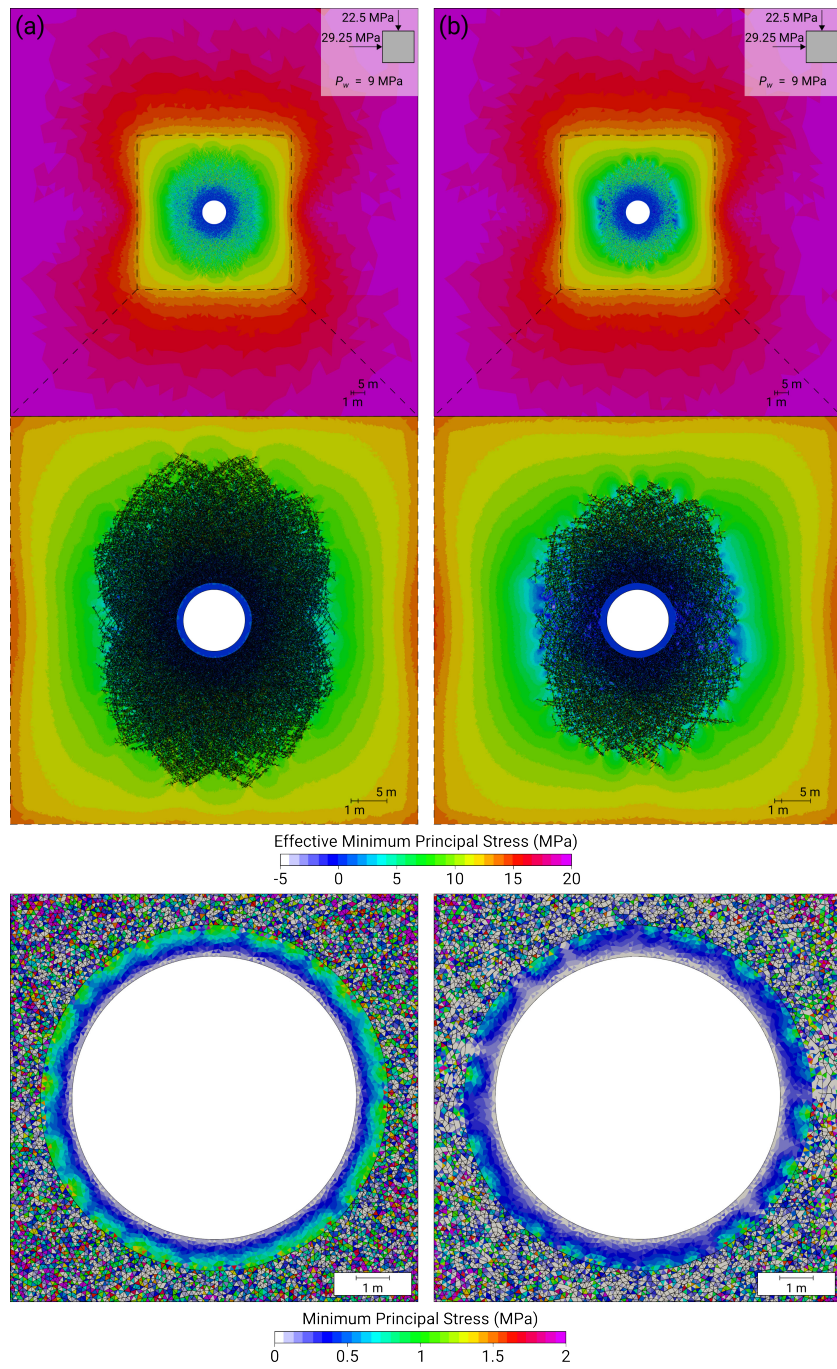


Fig. 6-20: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for shaft models with maximum horizontal in-situ stress ($\sigma_h = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

Positive stress is compression.

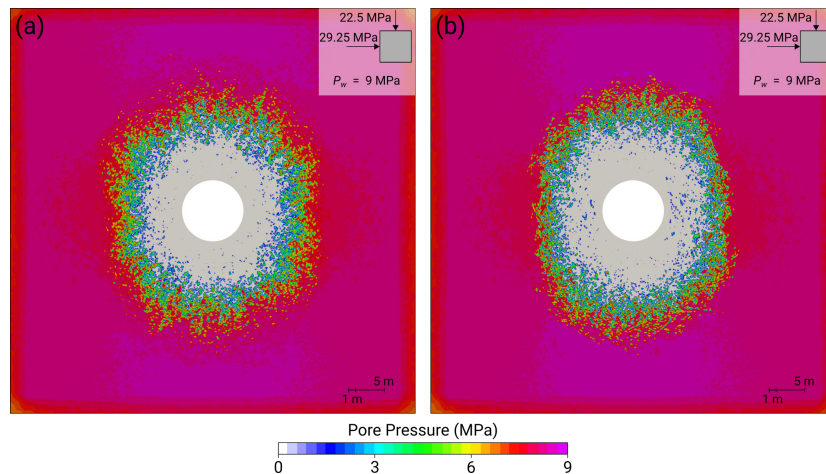


Fig. 6-21: Pore pressure contours for shaft models with maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for (a) lower bound and (b) upper bound strength calibrations

6.4 Effect of liner installation timing

Three shaft models with lower bound rock strength properties and in-situ stress ratios $\sigma_h/\sigma_H = 1$, $\sigma_h/\sigma_H = 0.77$, and $\sigma_H/\sigma_h = 1.3$ were simulated to assess the effect of increased α_s , and hence liner installation closer to the advancing face, on EDZ formation and excavation convergence (cases 7, 8 and 9). The objective of this analysis was to match the lower range of short-term excavation convergence provided by NAGRA (Fig. 3-8). For shaft models, the target convergence at any of the monitoring points was 6.2 cm which was approximately 1.2% of the shaft radius.

The Young's modulus reduction scheme for shaft models with minimum horizontal and isotropic in-situ stress (Fig. 6-22a) and maximum horizontal in-situ stress (Fig. 6-22b) are shown in Fig. 6-22. The stiffness of the core and liner materials were reduced from its original values over 0.4-0.5 M time steps (from 0.3 M to 0.7-0.8 M time steps) where it was held at the corresponding core softening ratio ($\alpha_s = 0.004$ for minimum horizontal and isotropic in-situ stress and $\alpha_s = 0.007$ for maximum horizontal in-situ stress) that resulted in approximately 6.2 cm (i.e., 1.2% of the shaft radius) of convergence at one of the four boundary monitoring points. The core was subsequently excavated at 1.35 M time steps and the liner was installed. The simulated temporal convergence and ground reaction curves are shown in Fig. 6-23, and as expected, the walls of the excavation converge as the Young's modulus of the core and liner were reduced. The anisotropy of the applied far-field stresses dictated the convergence anisotropy of the shaft. There was greater simulated wall convergence in the direction along the axis of σ_h than the axis of σ_H .

The simulated displacement contours and EDZ for this set of shaft models are shown in Fig. 6-24. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). After installing the stiff liner, there was up to 0.3 cm of additional convergence simulated. Compared to the results of the cases outlined in Sections 6.1 to 6.3, there was greater liner displacement due to a reduction in allowed pre-excavation convergence. The portion of the liner that exhibited the greatest amount of displacement were in the directions of σ_h while for the case of isotropic in-situ stress liner displacement was fairly uniform.

The extent of the EDZ for the case of minimum horizontal in-situ stress (Fig. 6-24a) was 15.0 m in the direction of σ_h and 8.1 m in the direction of σ_H of the excavation. The overall shape of the EDZ can be described as eye-shaped that was elongated in the direction of σ_h . The simulated EDZ contains conjugate shear fractures all around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) were formed approximately a depth of 5 m from $P_{0/180}$ and 7.3 m from $P_{90/270}$.

The extent of the EDZ for the case of isotropic in-situ stress (Fig. 6-24b) was 11.5 m around the shaft boundary. The overall shape of the EDZ can be described as a circle. The simulated EDZ contains conjugate shear fractures around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) were formed approximately a depth of 6.5 m from the excavation boundary

The extent of the EDZ for the case of maximum horizontal in-situ stress (Fig. 6-24c) was 12.2 m in the direction of σ_h and 6.7 m in the direction of σ_H of the excavation. The overall shape of the EDZ can be described as eye-shaped that was elongated in the direction of σ_h . The simulated EDZ contains conjugate shear fractures around the excavation, with a very dense fracture network of discrete blocks that are approximately one element size (i.e., $h = 0.15$ m) were formed approximately a depth of 4.5 m from $P_{0/180}$ and 7 m from $P_{90/270}$.

The aperture and slip of the fractures for the simulations are shown in Fig. 6-25. As observed in the plots, the region near the excavation boundary has the greatest aperture and slip, with slight bias in the direction of σ_h for the case of anisotropic in-situ stress fields. The depth of this zone from the shaft boundary was approximately 3.0-4.3 m into the rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. 6-26 and Fig. 6-27. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures, particularly in the direction of σ_h . There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. 6-28). As seen in Fig. 6-26a-b, simulated tangential stresses in the liner were fairly uniform at about 10 MPa. For the case of maximum horizontal in-situ stress (Fig. 6-26c), there was significantly greater tangential stresses in the liner, up to 20 MPa, which was a result of liner installation closer to the advancing face (i.e., greater α_s).

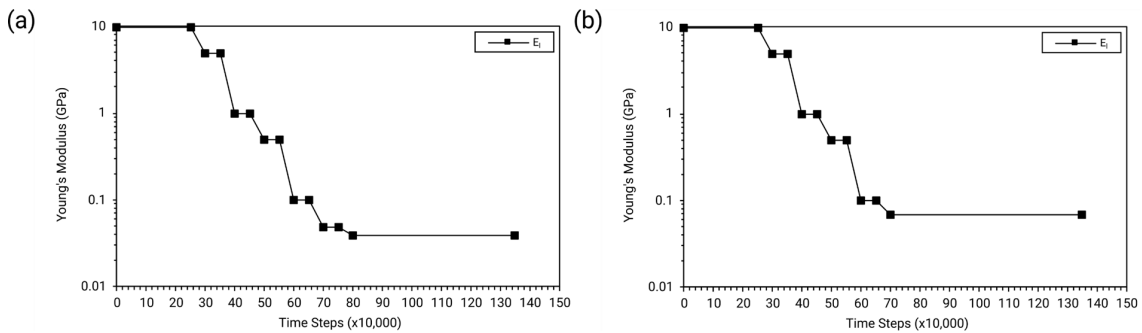


Fig. 6-22: The temporal softening scheme of the excavation core for shaft models with (a) minimum horizontal ($\sigma_h = 17.5$ MPa) and isotropic in-situ stress ($\sigma_h = 22.5$ MPa) and (b) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations

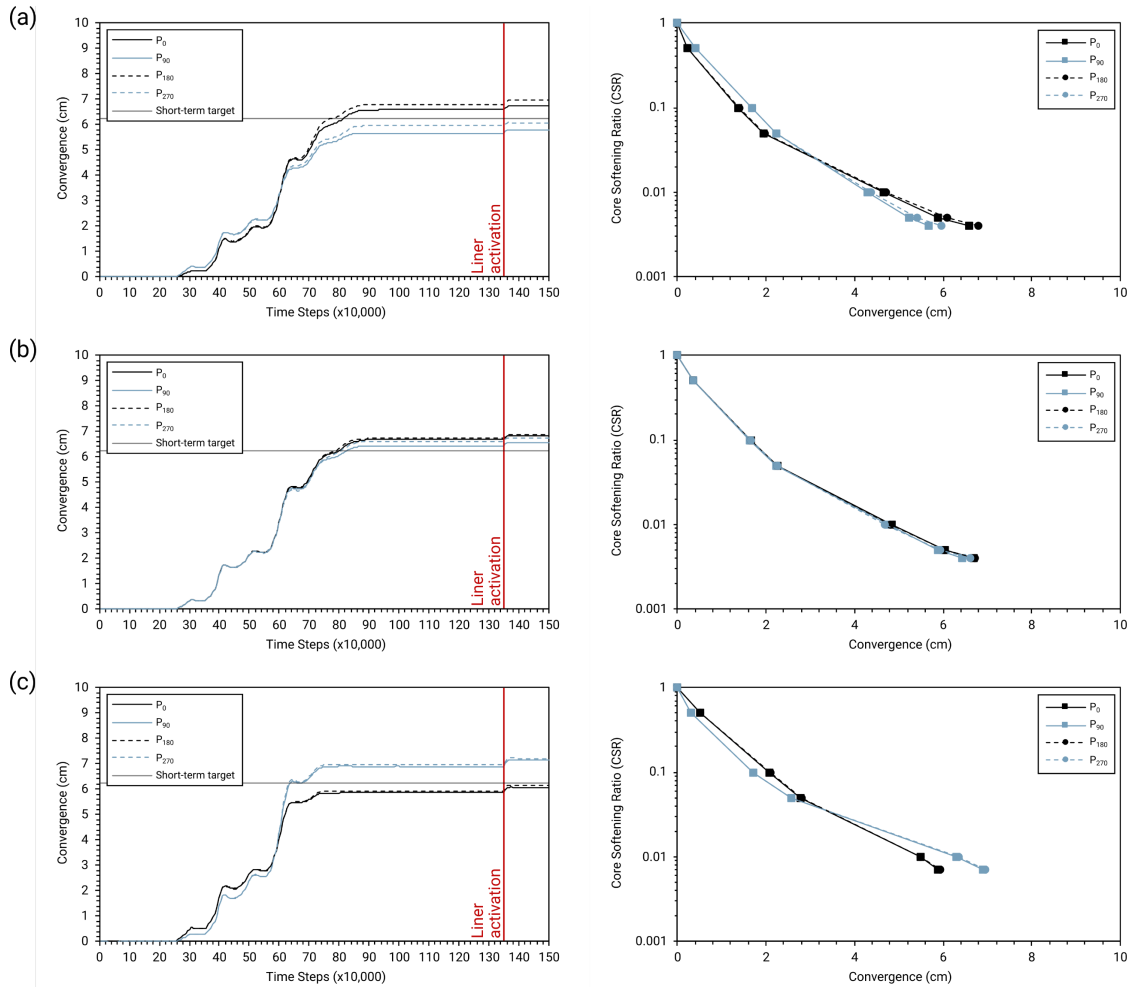


Fig. 6-23: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for shaft models with

(a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. For reference, the provided short-term sidewall and crown convergences by NAGRA are plotted as horizontal lines. Positive convergence is towards the centre of the excavation

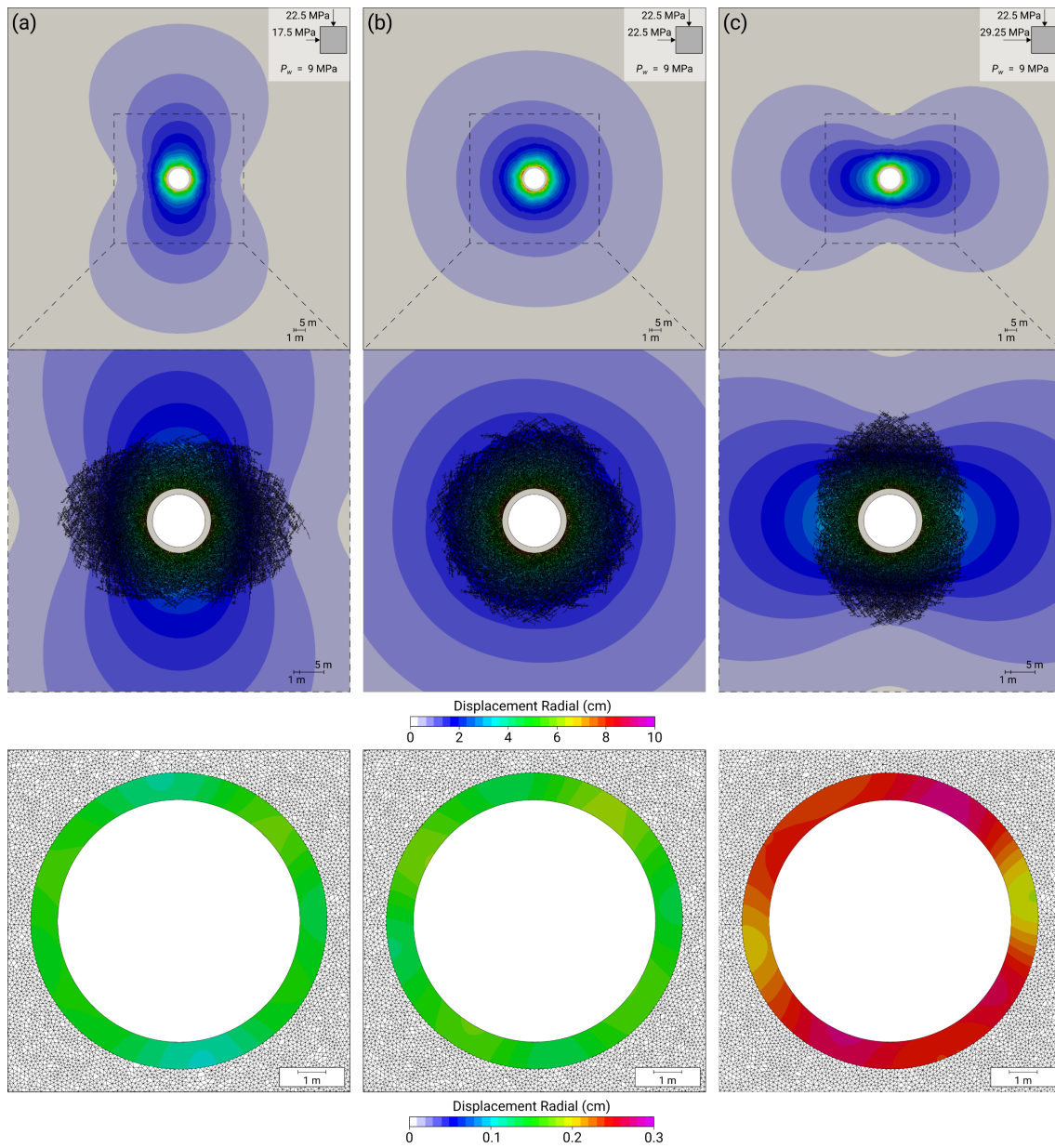


Fig. 6-24: Radial displacement contours and associated EDZ fracture pattern for shaft models with

(a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. The liner displacement shown is the net displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

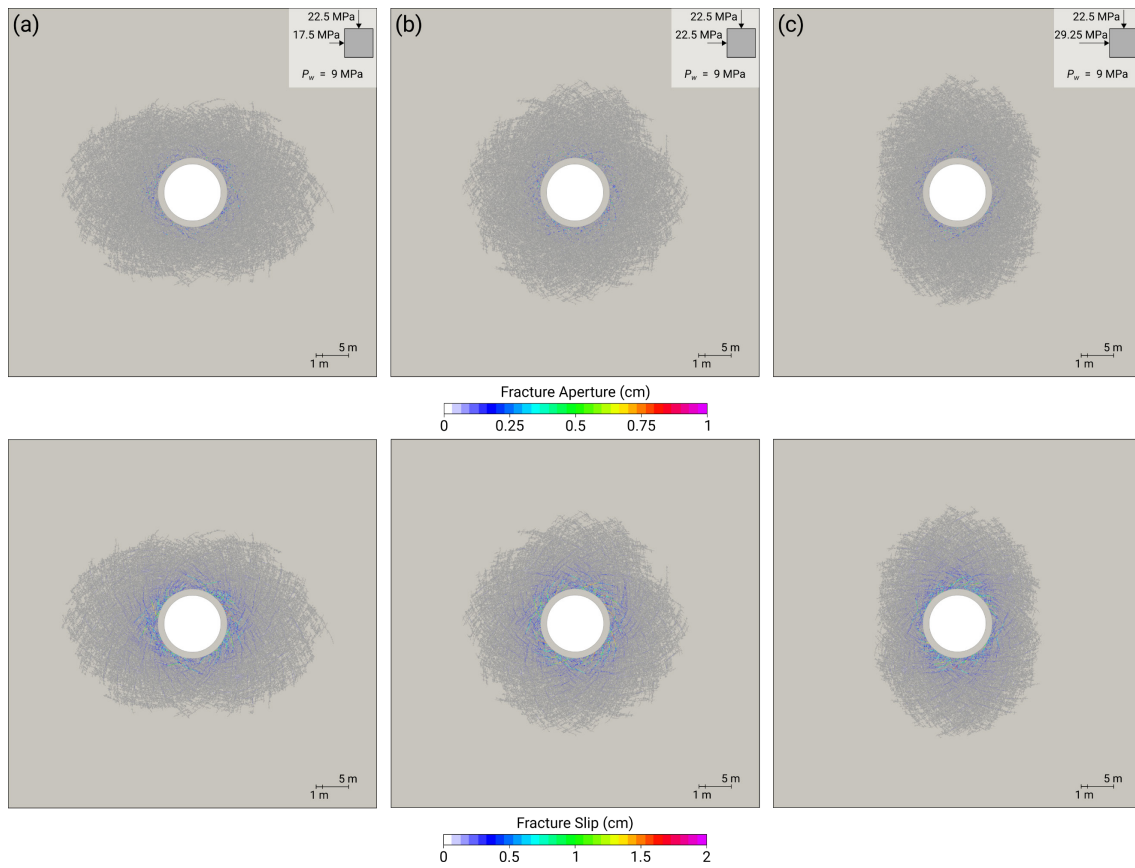


Fig. 6-25: Fracture aperture (top) and slip (bottom) contours for shaft models with
 (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations

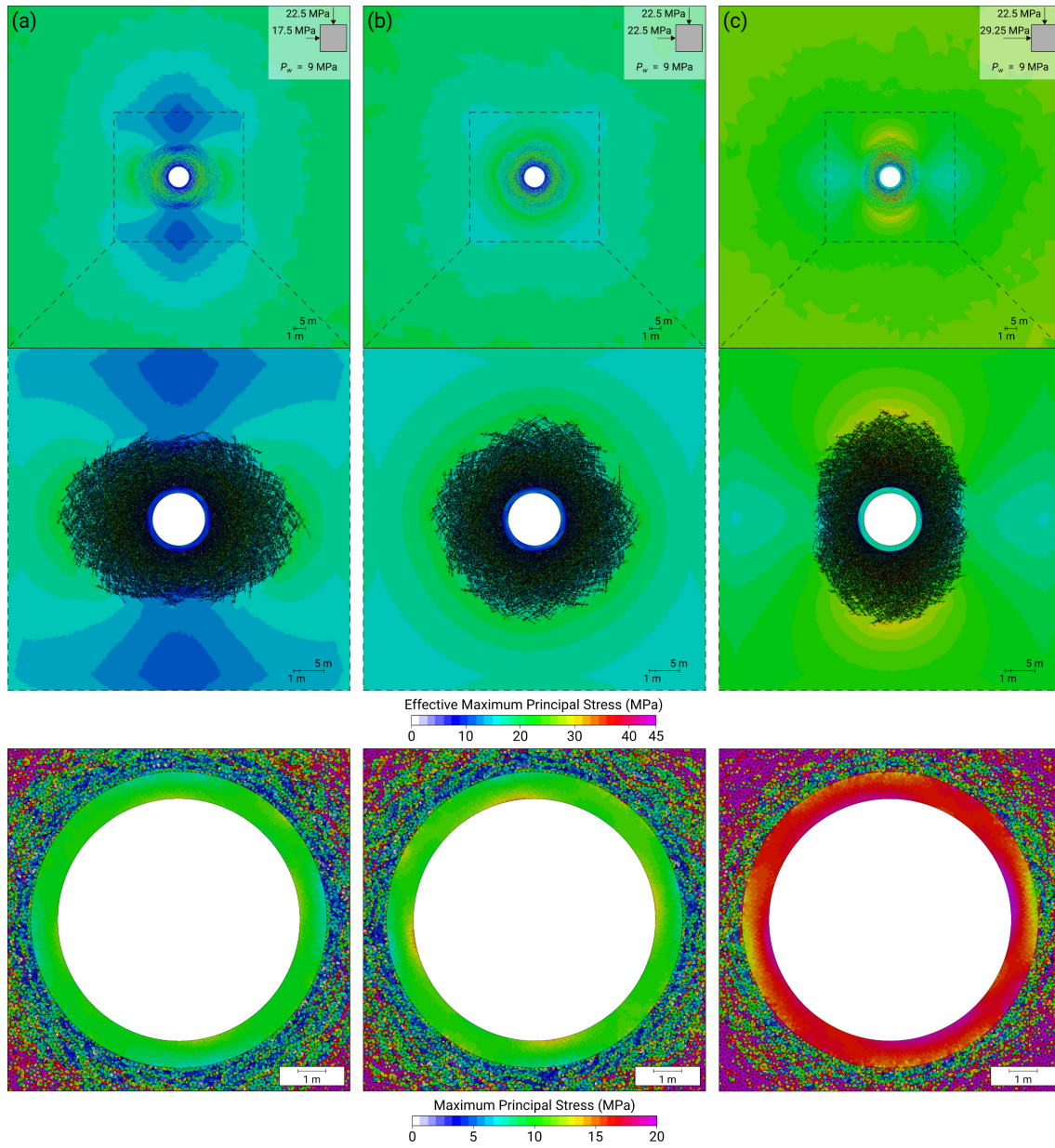


Fig. 6-26: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for shaft models with

(a) minimum horizontal in-situ stress ($\sigma_h = 17.5 \text{ MPa}$), (b) isotropic in-situ stress ($\sigma_h = 22.5 \text{ MPa}$), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25 \text{ MPa}$) for lower bound strength calibrations. Positive stress is compression.

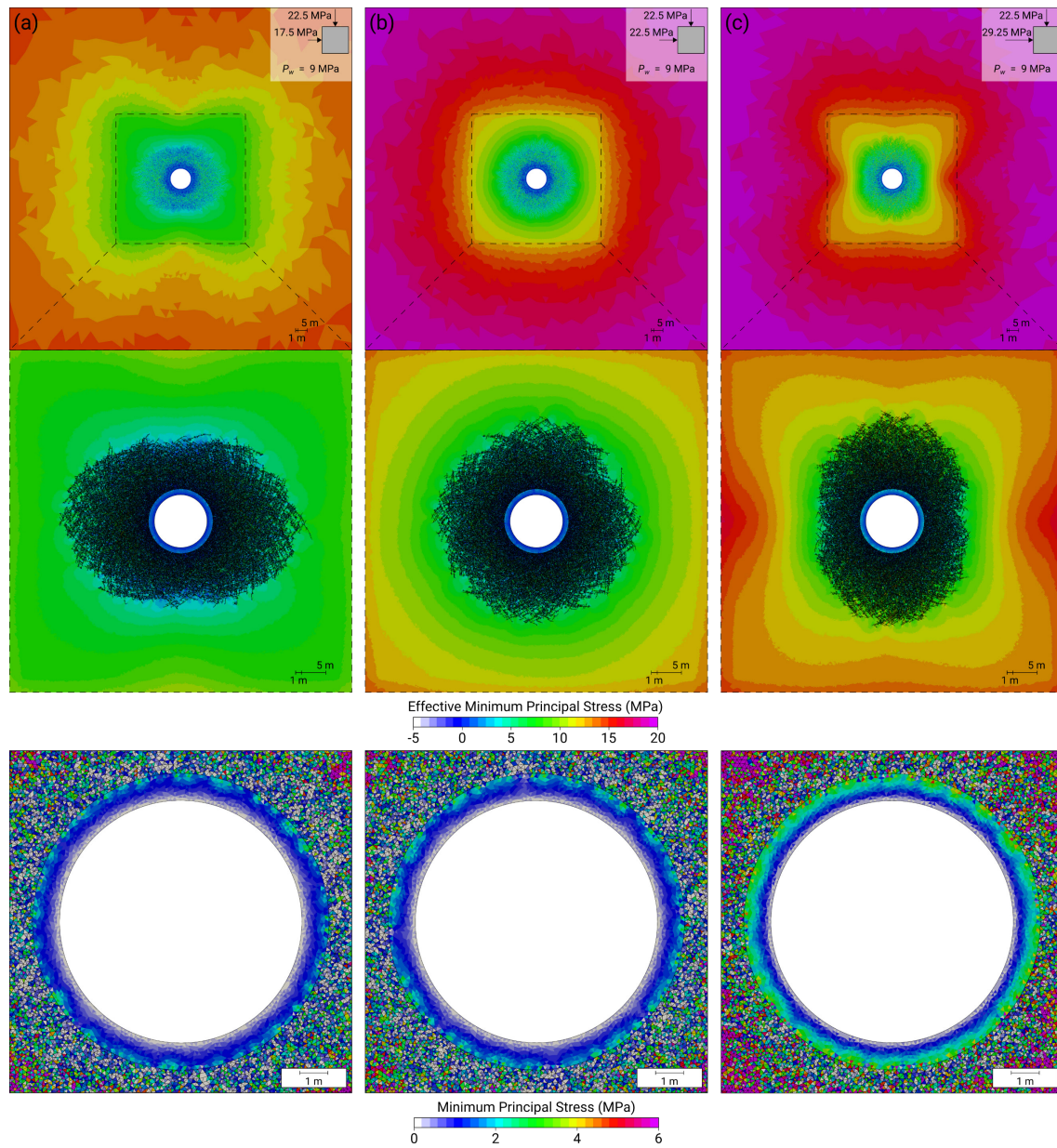


Fig. 6-27: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for shaft models with

(a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. Positive stress is compression

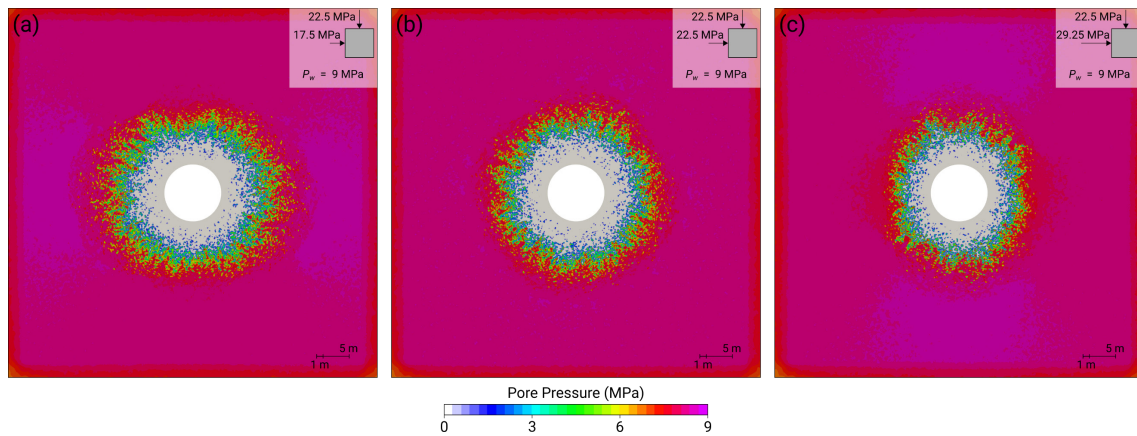


Fig. 6-28: Pore pressure contours for shaft models with
 (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations.

7 Summary and conclusions

The purpose of this study was to simulate, in 2D, the shape and extent of the EDZ for HLW emplacement tunnels, L/ILW emplacement caverns, and shaft designs at 900 m depth in the NL siting region. A sensitivity study on the rock strength, far-field stresses, and core softening regime and their effect on the simulated EDZ was performed. The overall simulation methodology as described in Geomechanica (2018) was adopted, and modified where applicable, and the latest available laboratory, field, and engineering design data were used. The simulation workflow in this study was altered as follows:

- The pore pressure applied to the models were one-way hydro-mechanically coupled, such that the pore pressure in the rock mass will decrease with an increase in fracture aperture as a constant pore pressure approach led to undesirable dynamic effects in the liner material.
- In a subset of cases simulated, different core softening ratios were applied to the principal directions of the excavation core to match the target GRC provided by NAGRA in terms of both the magnitude and location.

Summary of results

Simulations of representative EDZ for each of the designed cross-sections were conducted for three different stress regimes with horizontal-to-vertical ratios of 0.77, 1.0, and 1.3. The in-situ stress applied to the vertical axis of the model was constant for all simulations at 22.5 MPa. Furthermore, two sets of rock strength properties (lower and upper bound) representing the bounds of expected rock strength, were calibrated using the latest site-specific laboratory data documented in Madaschi et al. (2023). Additional simulations were performed using varying pre-excavation convergence targets, and hence core softening ratio, to study the effect of support installation timing and to match the provided GRC by NAGRA on EDZ formation. In total, nine cases were simulated for each cross-section. Summaries of the extents of the EDZ simulated under various conditions for HLW tunnel (Tab. 7-1), L/ILW cavern (Tab. 7-2), and shaft models (Tab. 7-3) are provided. The simulated displacement contours and EDZ for HLW tunnel (Fig. 7-1), L/ILW cavern (Fig. 7-2), and shaft models (Fig. 7-3) are also summarized.

Tab. 7-1: Summary of EDZ extent for HLW tunnel models

In-situ stress ratio, $k = \sigma_{xx}/\sigma_{yy}$	Rock strength properties	Core softening scheme	Core softening ratio, $\alpha_{sl} (\alpha_{st})$	Crown/Floor (m)	Sidewalls (m)
0.77	lower	isotropic	0.0025	4.5	5.7
0.77	upper	isotropic	0.0010	1.7	3.9
1.0	lower	isotropic	0.0020	5.5	4.8
1.0	upper	isotropic	0.0015	3.1	3.3
1.3	lower	isotropic	0.0040	5.0	3.2
1.3	upper	isotropic	0.0030	3.1	1.7
0.77	lower	anisotropic	0.0005 (0.04)	3.4	4.0
1.0	lower	anisotropic	0.0005 (0.04)	4.1	3.8
1.3	lower	anisotropic	0.0005 (0.05)	5.7	3.4

Note: If only one α_s value is stated, then $\alpha_s = \alpha_{sl} = \alpha_{st}$.

Tab. 7-2: Summary of EDZ extent for L/ILW cavern models

In-situ stress ratio, $k = \sigma_{xx}/\sigma_{yy}$	Rock strength properties	Core softening scheme	Core softening ratio, $\alpha_{sl} (\alpha_{st})$	Crown/Floor (m)	Sidewalls (m)
0.77	lower	isotropic	0.0025	13.7	19.9
0.77	upper	isotropic	0.001	8.1	15.9
1.0	lower	isotropic	0.003	17.6	18
1.0	upper	isotropic	0.00165	14.8	13.2
1.3	lower	isotropic	0.004	19.6	13.5
1.3	upper	isotropic	0.00375	14.7	10.5
0.77	lower	anisotropic	0.0005 (0.04)	12.4	18.7
1.0	lower	anisotropic	0.0005 (0.04)	18.1	16.2
1.3	lower	anisotropic	0.0008 (0.06)	20.2	14.4

Note: If only one α_s value is stated, then $\alpha_s = \alpha_{sl} = \alpha_{st}$.

Tab. 7-3: Summary of EDZ extent for shaft models

In-situ stress ratio, $k = \sigma_{xx}/\sigma_{yy}$	Rock strength properties	Core softening scheme	Core softening ratio, α_s	In x-direction (m)	In y-direction (m)
0.77	lower	isotropic	0.00040	21.1	13.0
0.77	upper	isotropic	0.00010	16.7	5.7
1.0	lower	isotropic	0.00045	16.0	16.0
1.0	upper	isotropic	0.00010	11.5	11.5
1.3	lower	isotropic	0.00095	10.0	16.2
1.3	upper	isotropic	0.00050	8.0	12.9
0.77	lower	isotropic	0.004	15.0	8.1
1.0	lower	isotropic	0.004	11.5	11.5
1.3	lower	isotropic	0.007	6.7	12.2

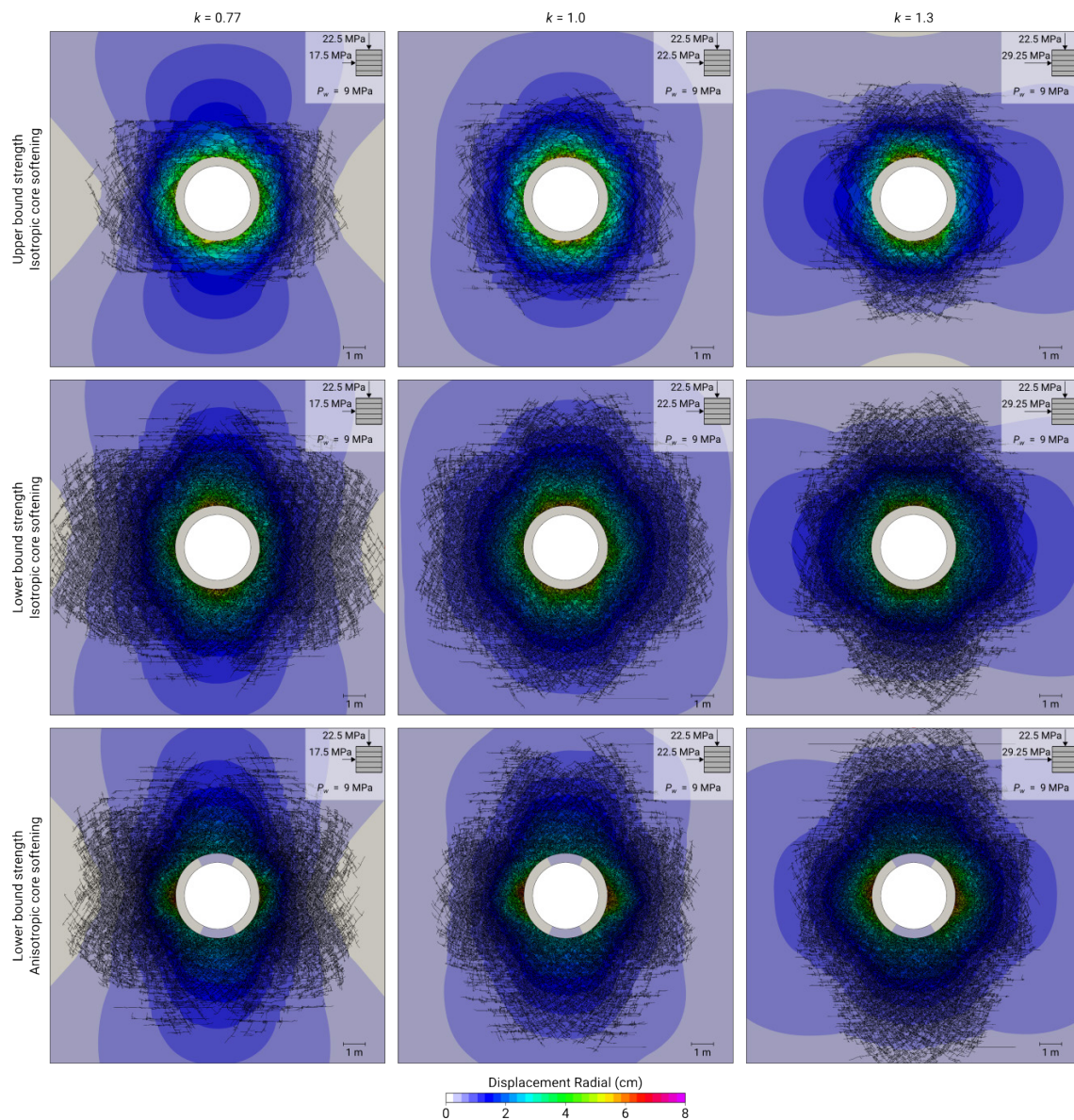


Fig. 7-1: Summary of radial displacement contours and associated EDZ fracture pattern for HLW tunnel models. The liner displacement shown is the net displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation

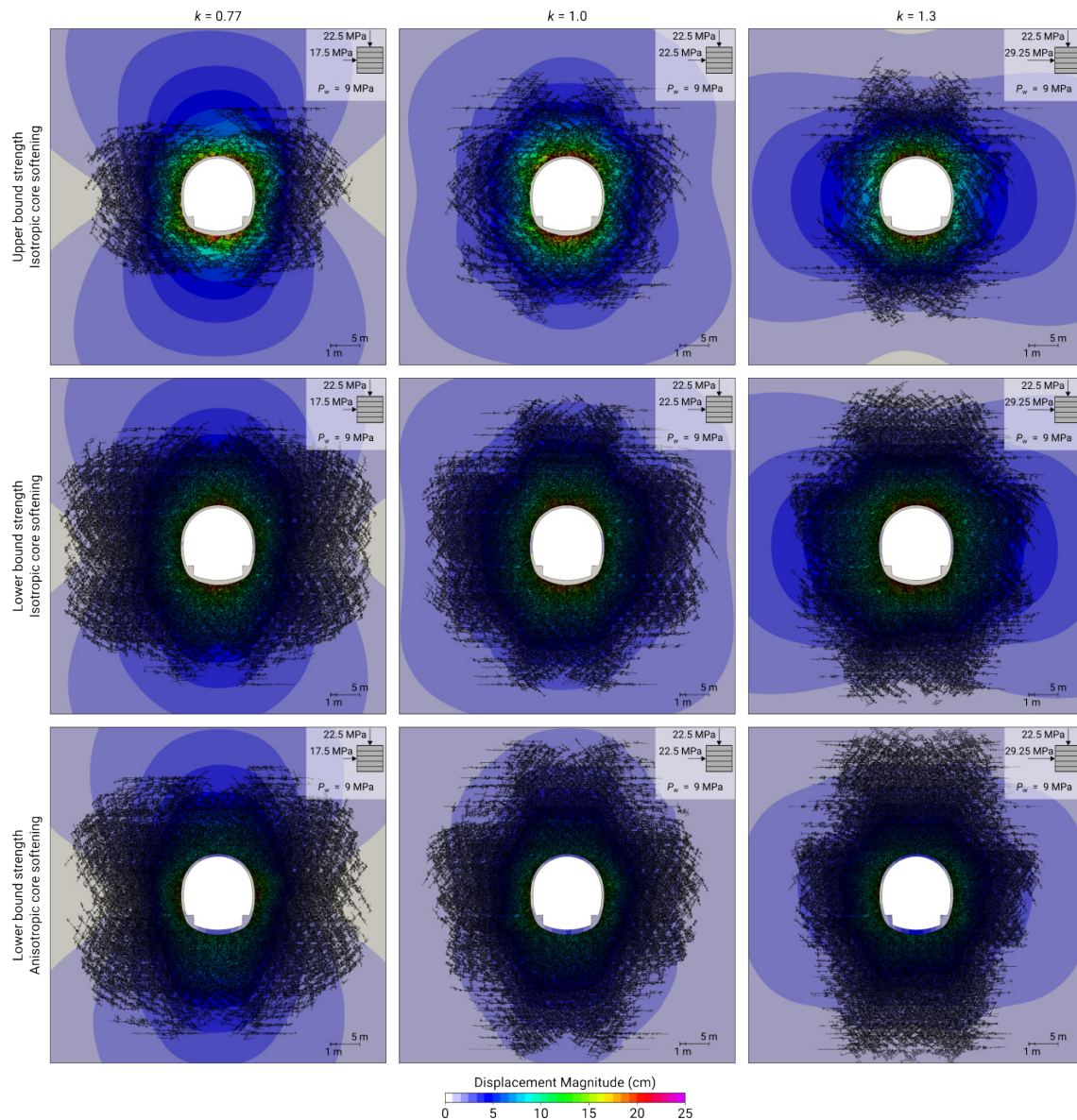


Fig. 7-2: Summary of displacement contours and associated EDZ fracture pattern for L/ILW caverns models. The liner displacement shown is the net displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

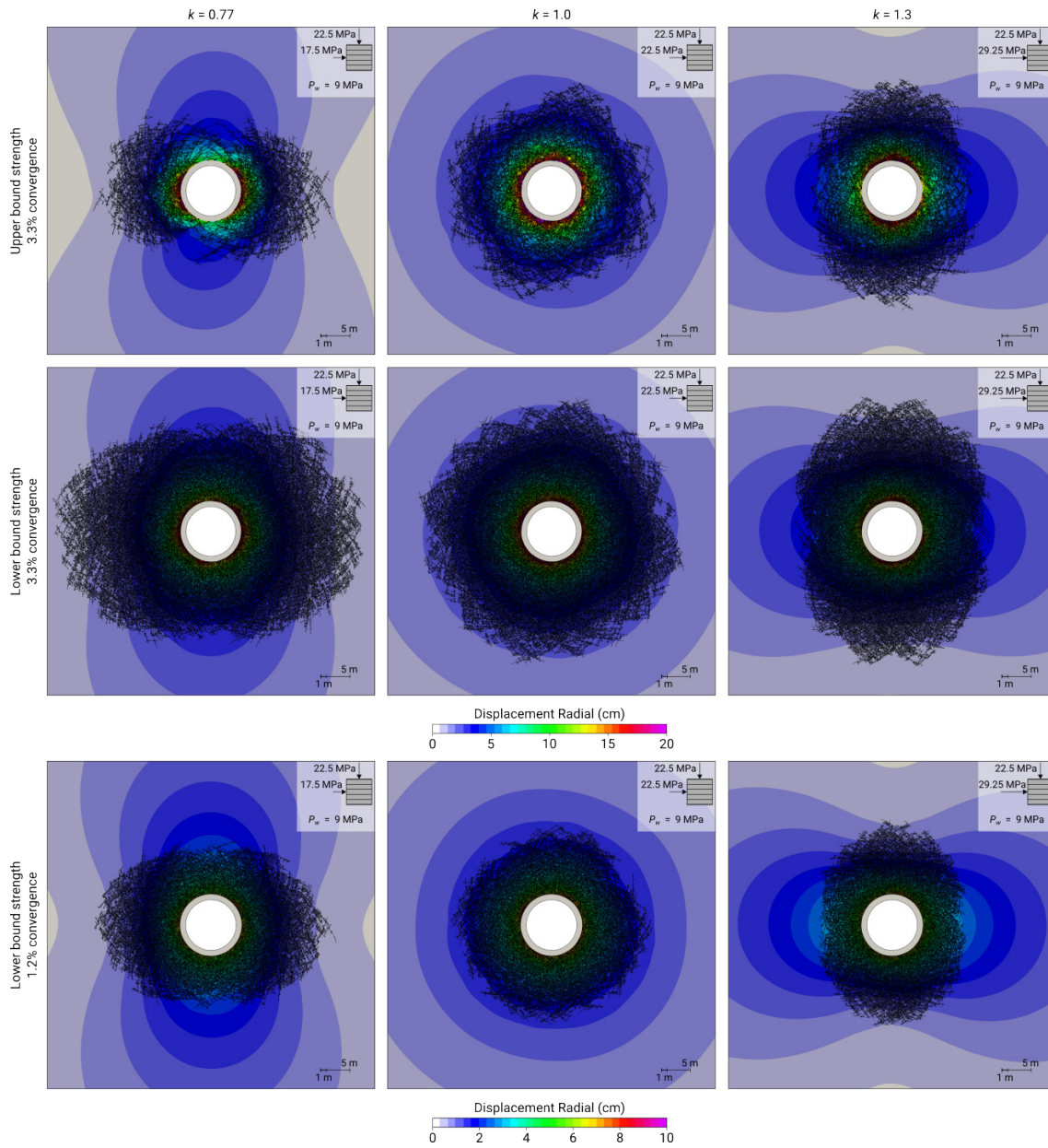


Fig. 7-3: Summary of radial displacement contours and associated EDZ fracture pattern for shaft models. The liner displacement shown is the net displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation.

Concluding remarks

Based on the results presented in this report, the following conclusions can be drawn for all cross-sections simulated:

- The simulated shape and extent of the EDZ was heavily influenced by the applied in-situ stress. In the case of isotropic in-situ stress, the shape and extent of the EDZ was fairly uniform around the excavation. However, when an anisotropic in-situ stress was applied, the major axis of the EDZ was parallel to the minor in-situ stress.

- Simulated near-excavation magnitude of slip and aperture was generally greater for the case of upper bound strength due to a sparse fracture network, and hence less interlocking compared to the case of lower bound strength. This region of large displacements correlates with a complete loss of pore pressure due to increases in fracture aperture.
- The range of studied rock strengths simulated had a significant effect on the simulated EDZ. In the case of weak rock, the overall extent of the EDZ was generally much larger and near the excavation boundary there was a zone of heavily fractured rock (blocks approximately one element size).
- Following the approach illustrated in Alcolea et al. (2014), the final EDZ fracture geometries, together with the corresponding stress states, produced in this study may be used as input for hydraulic simulations of the EDZ re-saturation process.

For HLW tunnel and L/ILW cavern models where the rock was simulated as a transversely isotropic material, the following observations were made:

- Rock mass failure was structurally controlled by bedding plane shearing and intra-bedding fracturing in the crown and floor of the excavation, while conjugate shearing was exhibited in the sidewalls.
- In cases with isotropic core softening regime, there was greater displacement simulated in the crown and floor of the excavation than the sidewalls due to the anisotropic stiffness of the rock and excavation core – where the rock was less stiff in the direction perpendicular (i.e., vertical axis) than parallel (i.e., horizontal axis) to bedding.
- In cases with anisotropic core softening regime, where the objective was to match the target GRC provided by NAGRA in terms of both the magnitude and location of convergence, resulted in undesired dynamic modelling effects post-excavation. Since the rock mass was softer and the excavation core was stiffer in the transverse direction, there was stored strain energy in the rock mass. Once the core was removed and the liner was installed, a sudden and drastic change in displacement was measured at the monitoring points resulting in unrealistically high liner strains and stresses.

For shaft models where the rock was simulated as an isotropic material, the following observations were made:

- Rock mass failure was predominantly in conjugate shear fractures all around the excavation.
- The displacement contrast between $P_{0/180}$ and $P_{90/270}$ was dictated by the applied in-situ stress field, where there was greater displacement simulated at the monitoring points coinciding with the minimum horizontal in-situ stress.
- For simulations cases where liner installation was performed at higher core softening ratios and wall convergence was 1.2% of the tunnel radius (i.e., support installed closer to the advancing face), there was approximately three times more simulated displacement, and hence tangential stress, in the liner.

The simulation results presented in this study are an update on the plausible range of EDZ that may form during the construction and operation of the combined repository. The models will be used to verify and compare against parallel numerical modelling studies on EDZ formation in Opalinus Clay, as well as validate their accuracy by comparing against field-scale experiments. In addition, it is anticipated that these results will be used to inform future design decisions and contribute to the permitting processes.

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App. A Investigating alternative modelling methodologies

A.1 Constant pore pressure

In the early iteration of simulations for this study, a constant pore pressure boundary condition was assigned to the rock mass. As described in Section 3.4, the pore pressure is applied as a surface pressure to the two adjoining edges of all finite element pairs (i.e., inside each crack element, see Fig. 3-6). In this approach, the pore pressure does not change as the EDZ develops (i.e., no drainage). Although this might be a conservative assumption, it is unrealistic to assume that the prescribed pore pressure (9 MPa) will maintain after the EDZ has developed since the aperture of the crack elements, and hence, fluid cavity volume, have increased. This approach has led to unintended dynamic liner effects upon core excavation in the L/ILW models, and to a lesser extent, the HLW models.

In Fig. A-1, a comparison of the displacements at the monitoring points (Fig. 3-9b) of an L/ILW model using the same core softening ratio of 0.038 and two different pore pressure application approaches is shown. As observed in Fig. A-1a, when the core is excavated, there is a sudden spike in displacement, followed by cyclic behaviour when a constant pore pressure is prescribed to the rock mass. When the pore pressure is allowed to dissipate in response to the fracturing in the rock mass, as is the case in Fig. A-1b, the dynamic effects are eliminated, but there is significantly less displacement using the same core softening ratio. Hence, re-calibration of the core softening ratio is necessary to obtain the correct ground reaction before installing the liner.

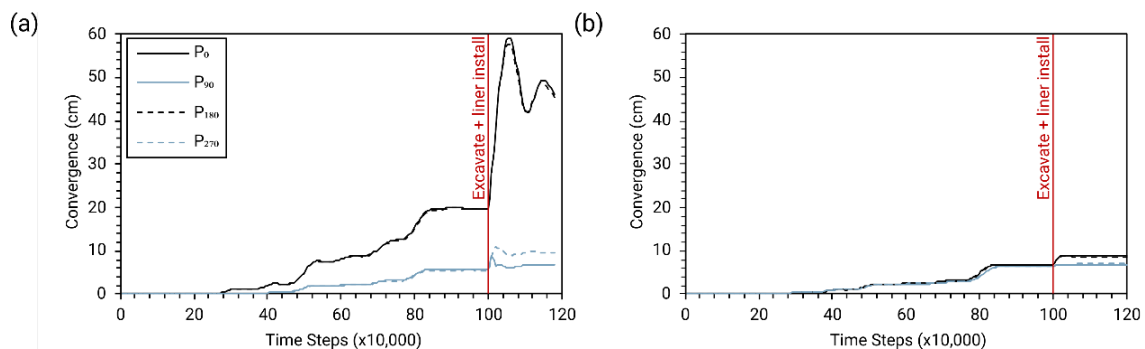


Fig. A-1: Displacement at monitoring points, where positive is towards the centre of the excavation, of an L/ILW simulation using (a) a constant pore pressure boundary condition and (b) one-way hydro-mechanical coupling approach, both using a core softening ratio of 0.038

For HLW models, there is similarly a drastic difference in simulated results when comparing the two pore pressure modelling approaches. In Fig. A-2, the displacements at the monitoring locations (Fig. 3-9a) are plotted against time for the case of an isotropic in-situ stress state and upper bound strength parameters. For comparative purposes, the pre-excavation displacements were matched between the constant and one-way coupled pore pressure approaches, which required a 17.5 times reduction in the core softening ratio (0.035 versus 0.002). Although the dynamic effects of the liner upon core excavation is relatively minor, there are significant differences in the overall EDZ that are formed (Fig. A-3). The shape of the EDZ in the one-way coupled hydromechanical approach is fairly uniform around the excavation and is highly influenced by the isotropic in-situ stress state. As opposed to the constant pore pressure approach with less fracturing and a horizontal EDZ. As fractures develop around the excavation, the pore pressure imposed on the rock mass enlarges a gap at one element depth into the rock mass from

the liner (Fig. A-4). As a result, the rock mass no longer deforms with the core, but instead a one element thick layer of rock elements, with the prescribed pore pressure, pushes towards the soft core (with zero pore pressure) causing it to deform and prematurely reach the target pre-excavation convergence. Consequently, a larger core softening ratio (0.035) is required to reach the target deformation and hence there is less fracturing in the rock mass.

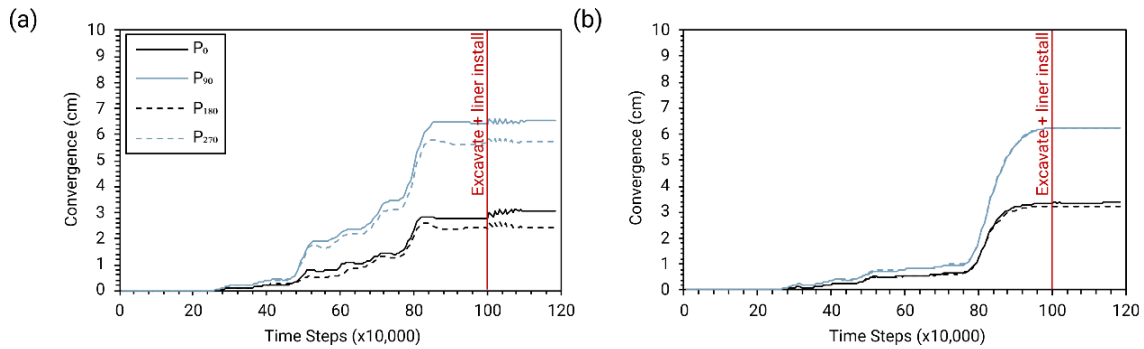


Fig. A-2: Displacement at monitoring points, where positive is towards the centre of the excavation, of an HLW simulation using (a) a constant pore pressure boundary condition with a core softening ratio of 0.035 and (b) one-way hydro-mechanical coupling approach with a core softening ratio of 0.002

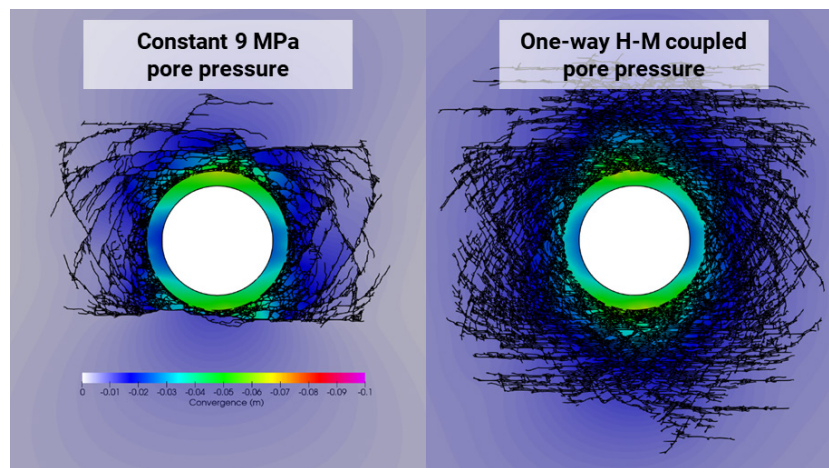


Fig. A-3: The EDZ formed after installation of the liner using the two different pore pressure modelling approaches

The pre-excavation convergence is matched between the two models.

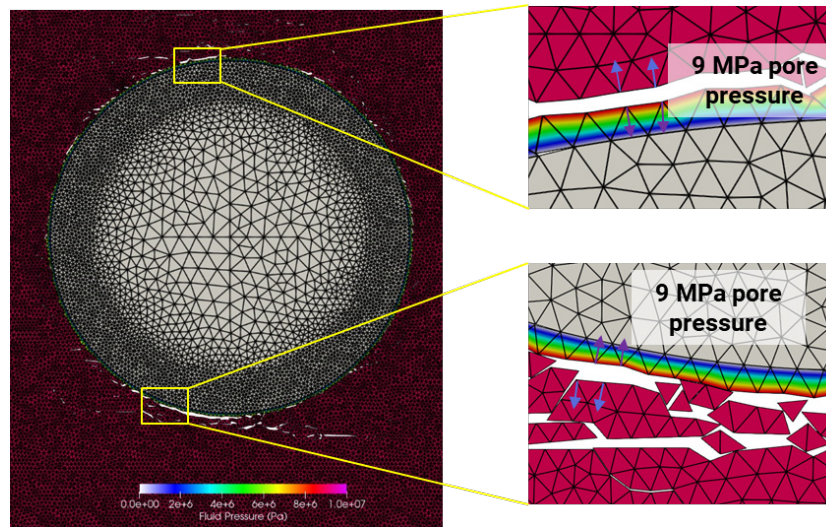


Fig. A-4: Application of constant pore pressure one element depth into the rock mass from the liner

A gap forming between the inactivated liner material as pore pressure acts towards the liner and towards the rock mass.

In conclusion, the constant pore pressure boundary condition was abandoned in this study to eliminate the undesirable dynamic effects observed in the liner, specifically in the L/ILW models. To maintain consistency in the overall study, HLW models were re-calibrated using the one-way coupled hydromechanical approach.

A.2 Force relaxation with material re-activation

The feasibility of a force relaxation with material re-activation excavation approach was assessed. This excavation method is analogous to the support pressure applied to the excavation boundary in studies such as (Nordas et al. 2024) and Nesol (2022). However, instead of applying a uniform support pressure to the excavation boundary, the support pressure applied to the excavation nodes is calculated as an unbalanced force vector, $\mathbf{F}_u$, at the excavation boundary nodes.

This approach was developed for Irazu to allow the application of support forces to the excavation boundary while also granting the ability to install a liner material that is explicitly meshed with triangular finite elements. The following operations take place (Fig. A-5) when the excavation sequence is initiated:

- The triangular finite elements that are marked for excavation (including the liner) are instantaneously, but temporarily, removed and hence, nodal deformation forces and contact interaction forces are no longer computed.
- The interior and exterior crack elements of the liner are set to broken and hence, no bonding forces are transmitted across the element edges.
- The unbalanced force vector, $\mathbf{F}_u$, at each of the nodes between the liner and rock are recorded and applied to those nodes.
- At each subsequent time step the liner elements are re-introduced and hence nodal deformation forces and contact interaction forces are computed.
- The unbalanced force vector is reduced using the force relaxation factor, α_f , defined for that time step as $(1-\alpha_f) \mathbf{F}_u$.

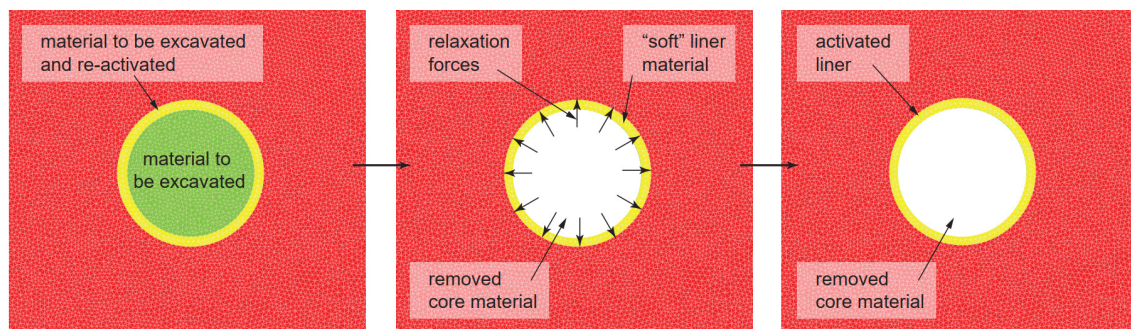


Fig. A-5: The stages of the force relaxation with material re-activation excavation method

Since the liner is explicitly incorporated into the mesh, it is necessary to reduce the Young's modulus of the liner to a small value such that it has negligible resistance and deforms with the rock mass. The liner material properties are then activated at a given α_f and the stress in the material is reset for the installation of a stress-free support. Subsequently, the α_f is reduced to zero to simulate further advancement of the excavation face. This approach is analogous to the one adopted in the FEA software ABAQUS (ABAQUS, 2004).

There were a couple of shortcomings that were identified with this approach through testing. They are:

- As fracturing in the rock near the liner-rock interface develops, elements can translate and rotate, which can result in the calculated unbalanced force vector to no longer be in the direction normal to the excavation boundary, thus leading to potential instability issues such as the one observed in the crown of Fig. A-6.
- Related to the first issue, this can result in significant localized deformation of the pre-installed liner leading to an unrealistic liner geometry when activated, as shown in Fig. A-6.

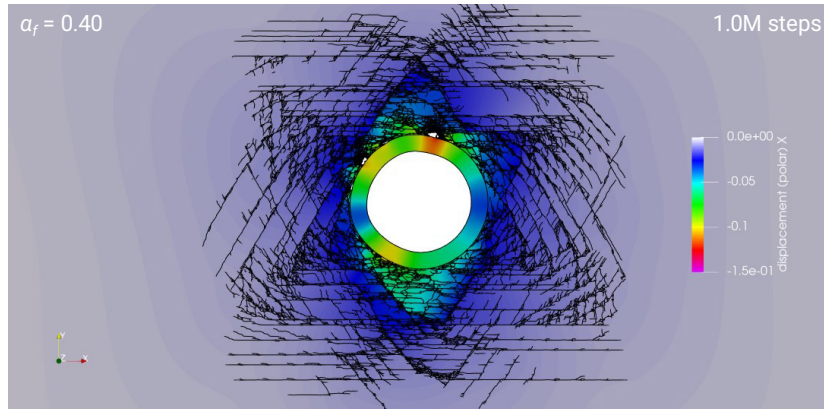


Fig. A-6: Significant liner deformation before activation using the force relaxation with material re-activation excavation approach in Irazu

As a result of these observations, the force relaxation with material re-activation excavation approach was abandoned for this study and the core modulus reduction method was used. It is a possibility these outlined issues are a consequence of the constant pore pressure boundary condition applied to the model, as described in App. A.1, but this was not further investigated.

A.3 Core softening schemes

A.3.1 HLW emplacement tunnel

Three HLW tunnel models with in-situ stress ratios $\sigma_h/\sigma_v = 1$, $\sigma_h/\sigma_v = 0.77$, and $\sigma_H/\sigma_v = 1.3$ and lower bound rock strength properties were simulated to assess the effect of prescribing different α_s values to E_l (i.e., α_{sl}) and E_t (i.e., α_{st}) on EDZ formation and excavation convergence (cases 7, 8 and 9). The objective of this analysis was to match as closely as possible the provided short-term GRC by NAGRA (Fig. 3-8) in terms of the location and magnitude of convergence. For HLW tunnel models, the target convergence at liner installation of the crown/floor and sidewalls were 2.2 cm and 6.1 cm.

From the results in Section 4, it was found that the anisotropic stiffness of the core, regardless of the anisotropy of the imposed far-field stresses in this study, had a dominant role in the deformation of the tunnel, where there was greater simulated convergence in the crown/floor of the excavation than the sidewalls. Hence, to achieve the convergence in the provided GRC (Fig. 3-8) it was necessary to apply an anisotropic core softening ratio that resulted in a core that was stiffer in the transverse direction than the lateral direction. The Young's modulus reduction scheme for HLW tunnel models for this analysis are shown in Fig. A-7. The transverse stiffness of the core and liner materials were reduced from its original values over 0.4-0.5 M time steps (from 0.3 M to 0.7-0.8 M time steps) where it was held at the corresponding α_{st} (as shown in Tab. 4-1) that resulted in approximately 2.2 cm (i.e., 1.2% of the tunnel radius) of convergence at monitoring points P₉₀ and/or P₂₇₀. The lateral stiffness of the core and liner materials were reduced from its original values over 0.9 M time steps (from 0.3 M to 1.2 M time steps) where it was held at the corresponding α_{sl} (as shown in Tab. 4-1) that resulted in approximately 6.1 cm (i.e., 3.3% of the tunnel radius) of convergence at monitoring points P₀ and/or P₁₈₀ (i.e., sidewalls). The core was subsequently excavated at 1.5 M time steps and the liner was installed.

The simulated temporal convergence and ground reaction curves are shown in Fig. A-8, as expected, the walls of the excavation converged as the Young's modulus of the core and liner were reduced. Upon core excavation and liner installation, there was a significant and sudden

change in convergence of up to 0.5 cm at the monitoring points. Since the transverse and lateral stiffness ($E_t = 5.8$ GPa; $E_l = 9.9$ GPa) contrast of the rock mass was greater than the contrast of the applied in-situ stress field, the excavation tended to deform more in the transverse or softer direction which coincides with the crown/floor of the tunnel. However, since the core stiffness in both directions were reduced over time using different α_s to control tunnel convergence, there was stored energy in the rock mass as the core was softened. Due to the dynamic nature of FDEM, once the core was removed, the energy stored in the rock mass was suddenly released leading to an undesired dynamic effect.

The simulated displacement contours and EDZ for HLW tunnel models with varying in-situ stress fields and lower bound strength are shown in Fig. A-9. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). As previously mentioned, there was significant deformation in the liner, where the dynamic release of energy in the rock mass upon core excavation resulted in up to 0.5 cm of convergence in the crown/floor and up to 0.4 cm of divergence in the sidewalls of the liner.

The extent of the EDZ for the case of minimum horizontal in-situ stress (Fig. A-9a) were 4.0 m from the sidewalls and 3.4 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon that was elongated in the direction of σ_h (i.e., sidewalls). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the sidewalls of the excavation. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a very dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.04$ m) were formed near the excavation. The depth of this dense fracture network was approximately 2.1 m from the sidewall and 1.7 m from the crown/floor of the excavation.

The extent of the EDZ for the case of isotropic in-situ stress (Fig. A-9b) were 3.8 m from the sidewalls and 4.1 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon. The EDZ had a fairly uniform depth all around the excavation and its overall shape was largely dictated by the applied isotropic in-situ stress field. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.04$ m) were formed near the excavation. The depth of this dense fracture network was approximately 1.8 m from the sidewalls and crown/floor of the excavation.

The extent of the EDZ for the case of maximum horizontal in-situ stress (Fig. A-9c) were 3.4 m from the sidewalls and 5.7 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon that was elongated in the direction of σ_h (i.e., crown/floor). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the crown/floor of the excavation. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a very dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.04$ m) were formed near the excavation. The depth of this dense fracture network was approximately 1.7 m from the sidewall and 2.1 m from the crown/floor of the excavation.

The aperture and slip of the fractures for the simulations are shown in Fig. A-10. As observed in the contour plots, the region near the excavation boundary in the sidewalls had the greatest aperture and slip, predominantly bedding plane slip in the haunch of the excavation and conjugate shearing in the sidewalls. This was a result of the prescribed anisotropic core stiffness that permitted greater displacement in the sidewalls. The depth of this zone from the HLW tunnel boundary was approximately 1.2-1.4 m into the rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. A-11 and Fig. A-12. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures, particularly in the direction of σ_h . There were regions of stress release near the excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. A-13). Due to the significant convergence and divergence of the liner in the crown/floor and sidewalls post-installation, the liner was seemingly highly stressed. There were tensile stresses up to 25 MPa in the crown/floor region of the liner, and tangential stresses up to 50 MPa in the sidewalls. The artificially high stresses in the liner were a result of the aforementioned unrealistic dynamic effects.

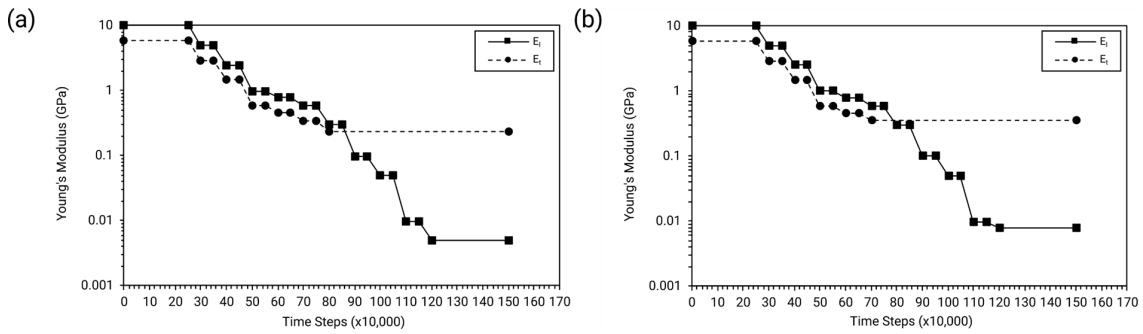


Fig. A-7: The temporal softening scheme of the excavation core for HLW tunnel models with (a) minimum horizontal ($\sigma_h = 17.5$ MPa) and isotropic in-situ stress ($\sigma_h = 22.5$ MPa) and (b) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations.

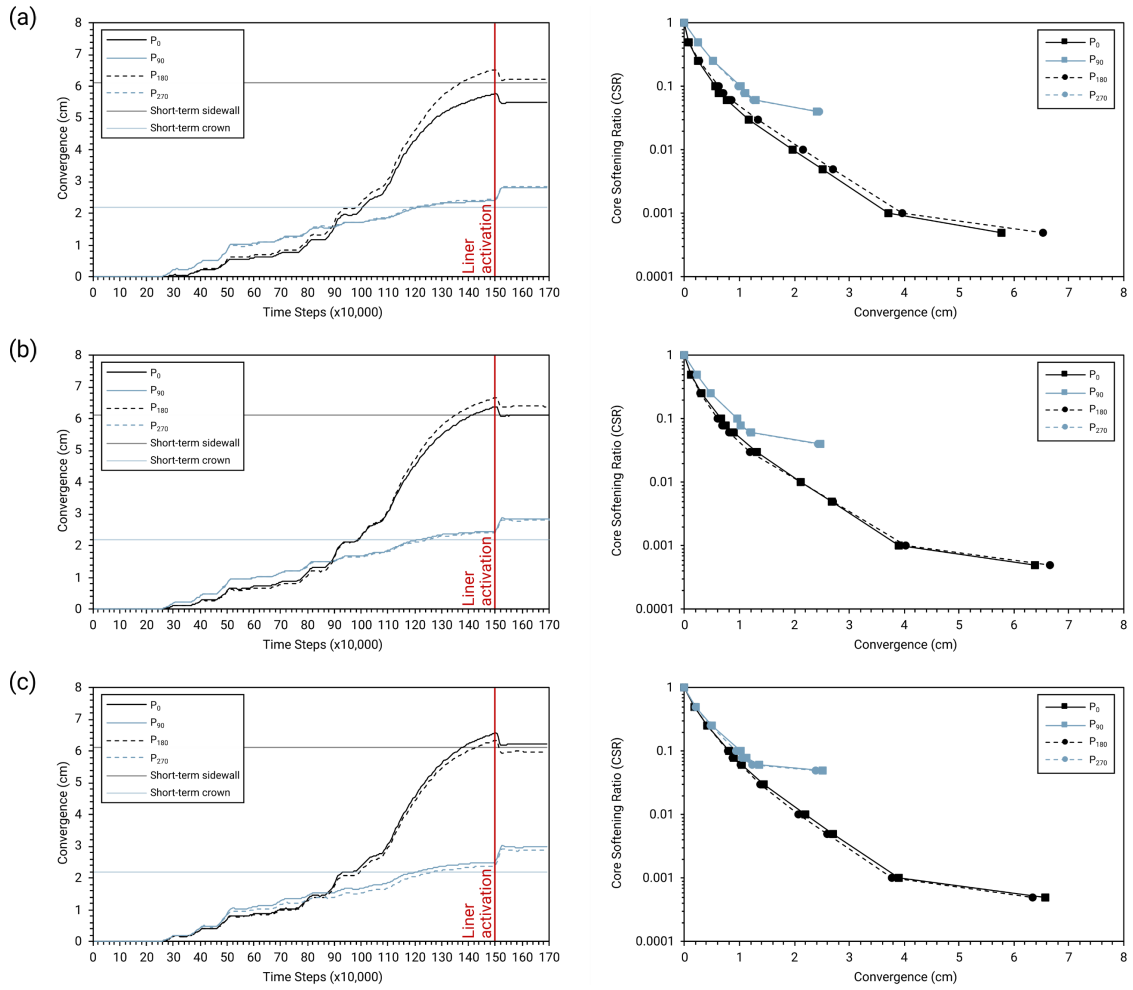


Fig. A-8: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. For reference, the provided short-term sidewall and crown convergences by NAGRA are plotted as horizontal lines. Positive convergence is towards the centre of the excavation

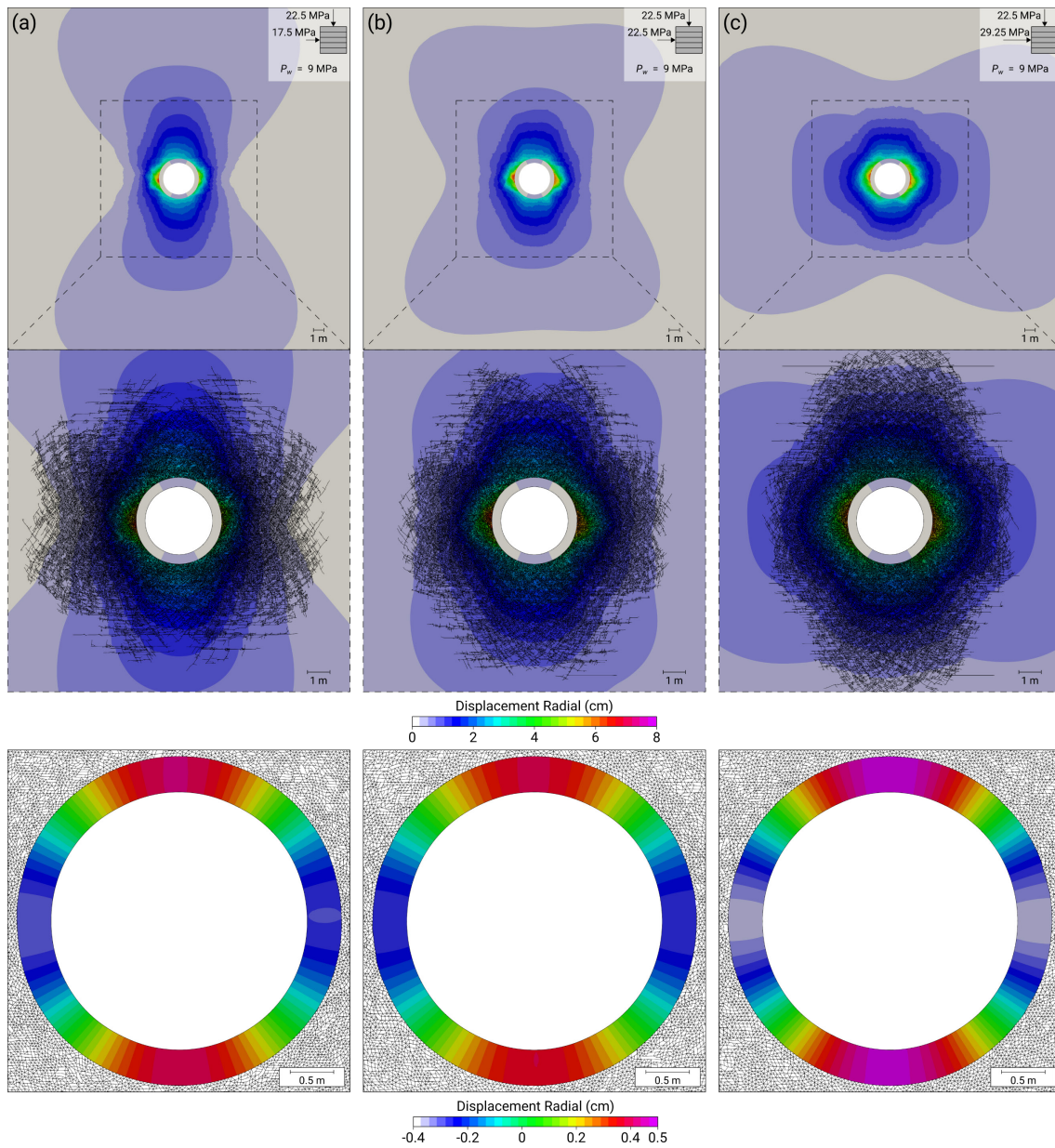


Fig. A-9: Radial displacement contours and associated EDZ fracture pattern for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. The liner displacement shown is the net displacement since the liner was installed. Positive radial displacement is towards the centre of the excavation

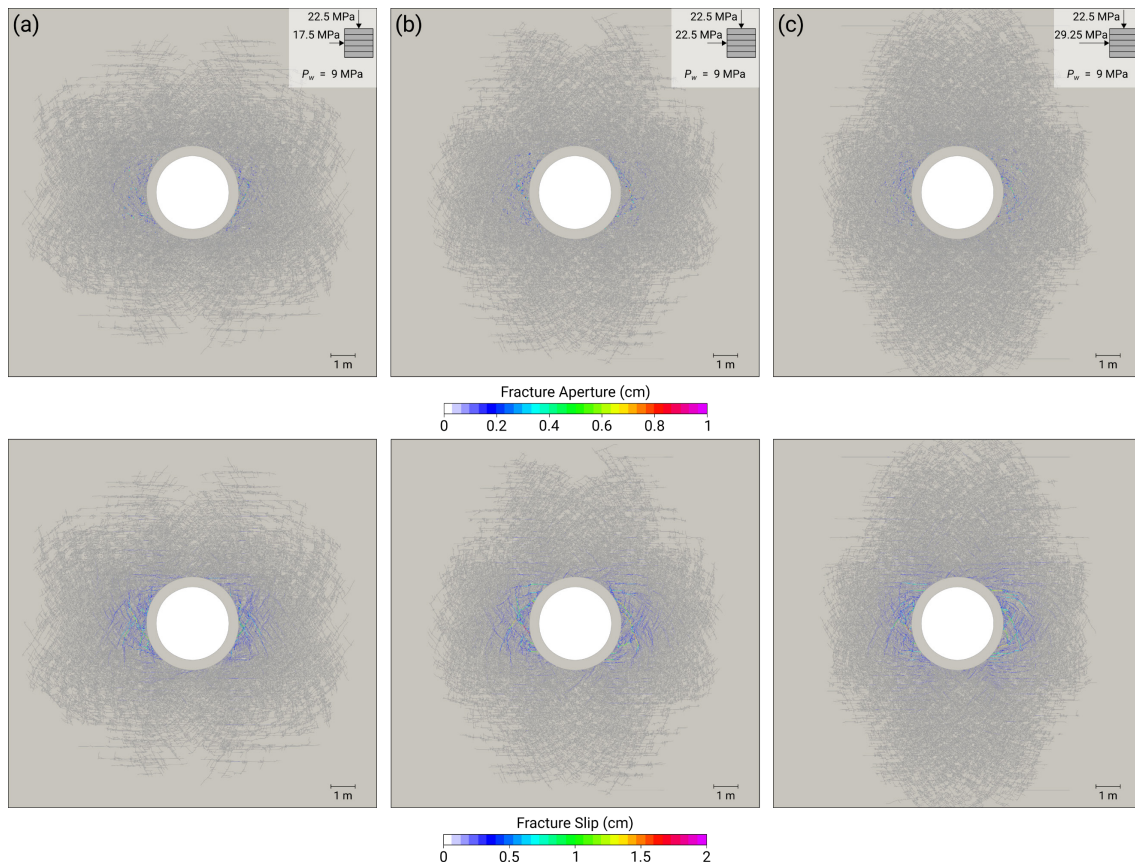


Fig. A-10: Fracture aperture (top) and slip (bottom) contours for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations.

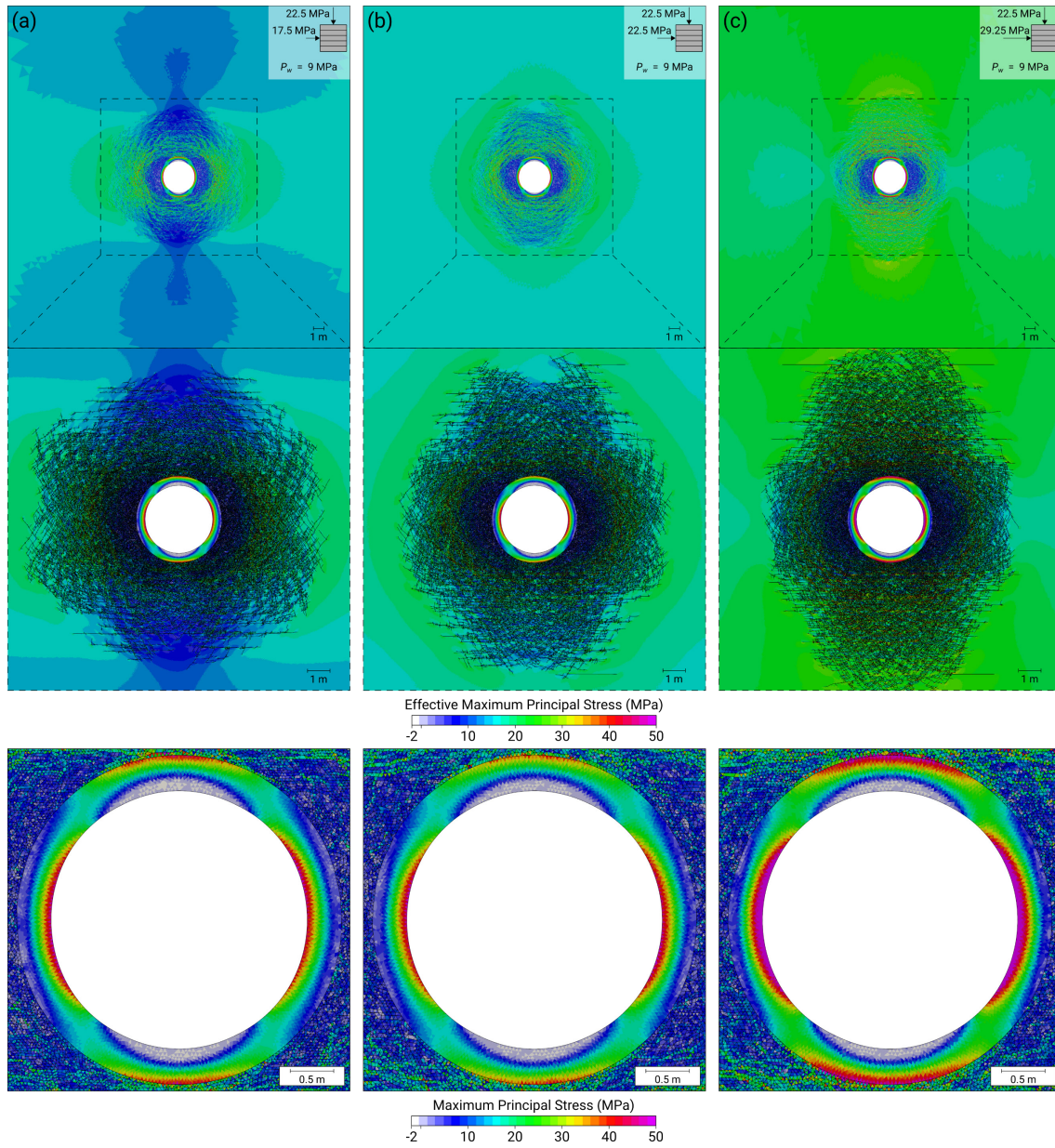


Fig. A-11: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. Positive stress is compression

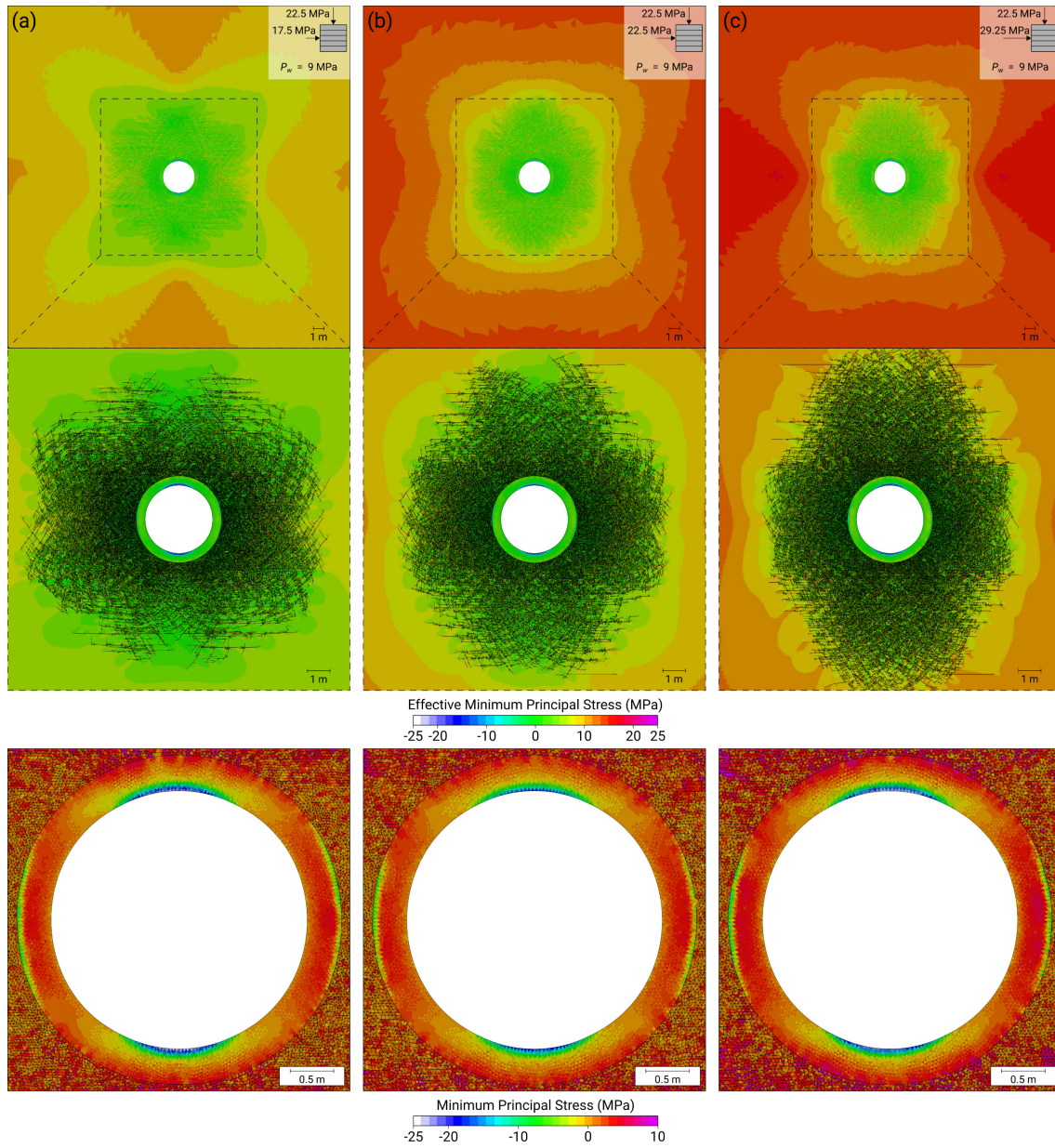


Fig. A-12: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. Positive stress is compression

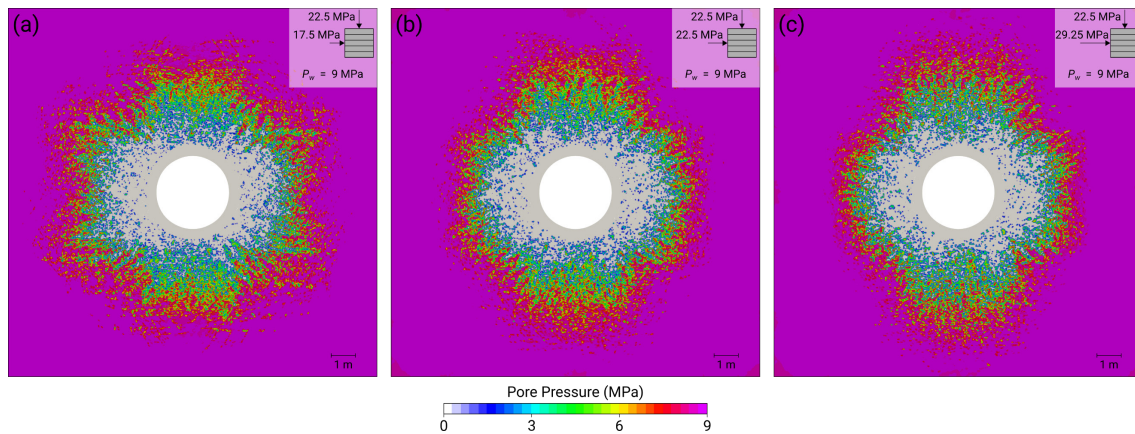


Fig. A-13: Pore pressure contours for HLW tunnel models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations

A.3.2 L/ILW emplacement cavern

Three L/ILW tunnel models with in-situ stress ratios $\sigma_h/\sigma_v = 1$, $\sigma_h/\sigma_v = 0.77$, and $\sigma_H/\sigma_v = 1.3$ and lower bound rock strength properties were simulated to assess the effect of prescribing different α_s values to E_l (i.e., α_{sl}) and E_t (i.e., α_{st}) on EDZ formation and excavation convergence (cases 7, 8 and 9). The objective of this analysis was to match as closely as possible the provided short-term GRC by NAGRA (Fig. 3-8) in terms of the location and magnitude of convergence. For L/ILW cavern models, the target convergence before liner installation of the crown/floor and sidewalls were 8.5 cm and 20.5 cm.

From the results in Section 5, it was found that the anisotropic stiffness of the rock mass, regardless of the anisotropy of the imposed far-field stresses in this study, had a dominant role in the deformation of the tunnel, where there was greater simulated convergence in the crown/floor of the excavation than the sidewalls. Hence, to achieve the convergence in the provided GRC (Fig. 3-8) it was necessary to apply an anisotropic core softening ratio that resulted in a core that was stiffer in the transverse direction than the lateral direction. The Young's modulus reduction scheme for L/ILW cavern models for this analysis are shown in Fig. A-14. The transverse stiffness of the core and liner materials were reduced from its original values over 0.5 M time steps (from 0.3 M to 0.8 M time steps) where it was held at the corresponding α_{st} (as shown in Tab. 5-1) that resulted in approximately 8.5 cm (i.e., 1.2% of half the excavation height) of convergence at monitoring points P_{90} and/or P_{270} . The lateral stiffness of the core and liner materials were reduced from its original values over 0.9 M time steps (from 0.3 M to 1.2 M time steps) where it was held at the corresponding α_{sl} (as shown in Tab. 5-1) that resulted in approximately 20.5 cm (i.e., 3.3% of half the excavation span) of convergence at monitoring points P_0 and/or P_{180} (i.e., sidewalls). The core was subsequently excavated at 1.5 M time steps and the liner was installed.

The simulated temporal convergence and ground reaction curves are shown in Fig. A-15, as expected, the walls of the excavation converged as the Young's modulus of the core and liner were reduced. Upon core excavation and liner installation, there was a significant and sudden change in convergence of approximately 1-3 cm at the monitoring points. Since the transverse and lateral stiffness ($E_t = 5.8$ GPa; $E_l = 9.9$ GPa) contrast of the rock mass was greater than the contrast of the applied in-situ stress field, the excavation tended to deform more in the softer direction which coincides with the crown/floor of the cavern. However, since the core stiffness in both directions were reduced over time using different α_s to control tunnel convergence, there was

stored energy in the rock mass as the core was softened. Due to the dynamic nature of FDEM, once the core was removed, the energy stored in the rock mass was suddenly released leading due to an undesired and unrealistic dynamic effect.

The simulated displacement contours and EDZ for L/ILW cavern models with varying in-situ stress fields and lower bound strength are shown in Fig. A-16. The liner displacement shown was the difference in convergence from the time of liner installation to the shown simulation result (i.e., liner displacements are zeroed upon liner activation). As previously mentioned, there was significant deformation in the liner, where the dynamic release of energy upon core excavation resulted in up to 5 cm of convergence in the crown/floor and relatively negligible convergence in the sidewalls of the liner.

The extent of the EDZ for the case of minimum horizontal in-situ stress (Fig. A-16a) were 18.7 m from the sidewalls and 12.4 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon that was elongated in the direction of σ_h (i.e., sidewalls). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the sidewalls of the excavation. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a very dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.15$ m) were formed near the excavation. The depth of this dense fracture network was approximately 9.5 m from the sidewall and 7.8 m from the crown/floor of the excavation.

The extent of the EDZ for the case of isotropic in-situ stress (Fig. A-16b) were 16.2 m from the sidewalls and 18.1 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon that was slightly elongated in the vertical direction. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.15$ m) were formed near the excavation. The depth of this dense fracture network was approximately 9.3 m from the sidewalls and crown/floor of the excavation.

The extent of the EDZ for the case of maximum horizontal in-situ stress (Fig. A-16c) were 14.4 m from the sidewalls and 20.2 m from the crown/floor of the excavation. The overall shape of the EDZ could be described as an octagon that was elongated in the direction of σ_h (i.e., crown/floor). The elongation of the EDZ in the direction of σ_h was a result of the applied anisotropic in-situ stress field and resulting stress redistribution from the excavation process whereby the areas of greatest compressive stress were at the crown/floor of the excavation. The simulated EDZ contained bedding parallel shearing and intra-bedding fracturing above/below the crown/floor and conjugate shear fractures in the sidewalls. As these fractures intersected and coalesced, a very dense fracture network emerged and discrete blocks that were approximately one element size (i.e., $h = 0.15$ m) were formed near the excavation. The depth of this dense fracture network was approximately 8.3 m from the sidewall and 12.8 m from the crown/floor of the excavation.

The aperture and slip of the fractures for the simulations are shown in Fig. A-17. As observed in the plots, the region near the excavation boundary in the sidewalls has the greatest aperture and slip, predominantly bedding plane slip in the haunch of the excavation and conjugate shearing in the sidewalls. This was a result of the prescribed anisotropic core stiffness that allowed for greater displacement in the sidewalls before excavation. The depth of this zone from the L/ILW cavern sidewall was approximately 4.5-6.0 m into the rock mass.

The maximum and minimum principal stress contours for the simulations are shown in Fig. A-18 and Fig. A-19. Regions of greatest compressive stress were at the fracture tips of conjugate shear fractures, particularly in the direction of σ_h . There were regions of stress release near the

excavation boundary correlating with areas of high fracture density, slip, and aperture. Within this region, due to the increase in aperture from fracturing the pore pressure had effectively reduced from 9 MPa to zero (Fig. A-20). Due to the aforementioned dynamic effects triggered by excavation of a highly anisotropic core, tangential stresses up to 100 MPa were simulated in the liner.

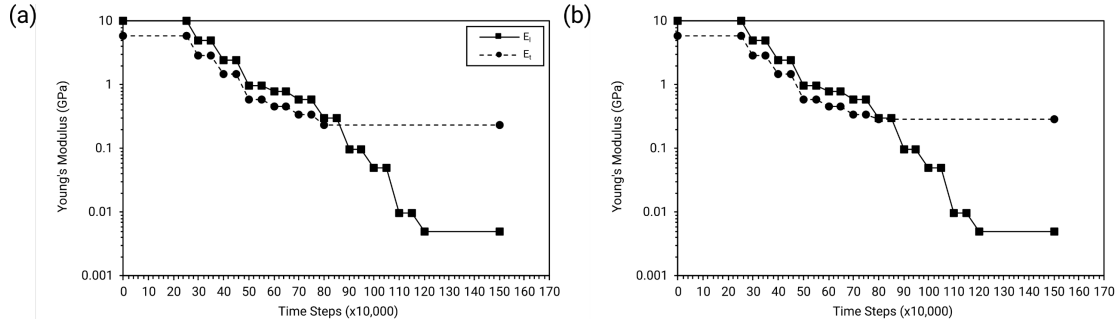


Fig. A-14: The temporal softening scheme of the excavation core for L/ILW cavern models with (a) minimum horizontal ($\sigma_h = 17.5$ MPa) and isotropic in-situ stress ($\sigma_h = 22.5$ MPa) and (b) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations

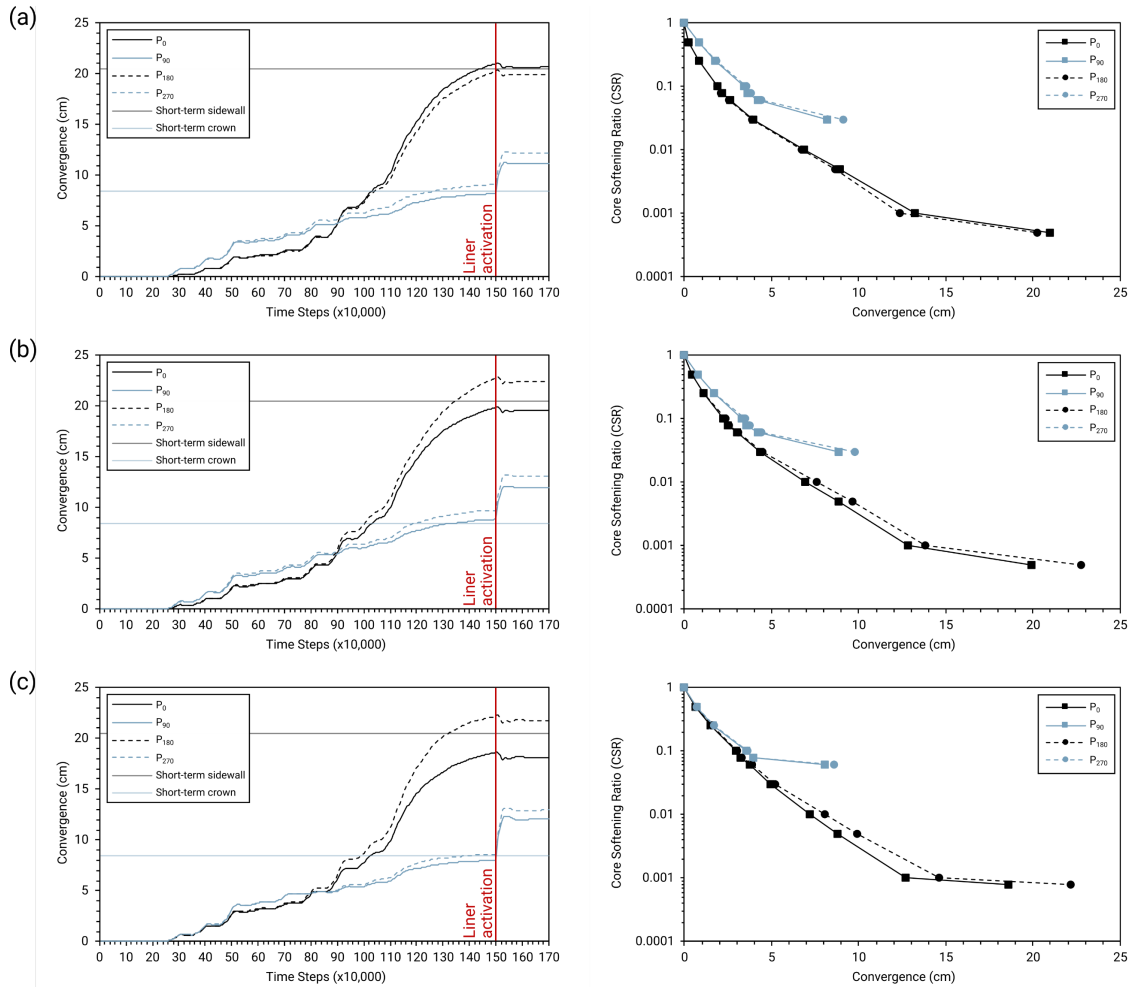


Fig. A-15: The temporal convergence (left) and ground reaction curves (right) at the four monitoring points are shown for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. For reference, the provided short-term sidewall and crown convergences by NAGRA are plotted as horizontal lines. Positive convergence is towards the centre of the excavation

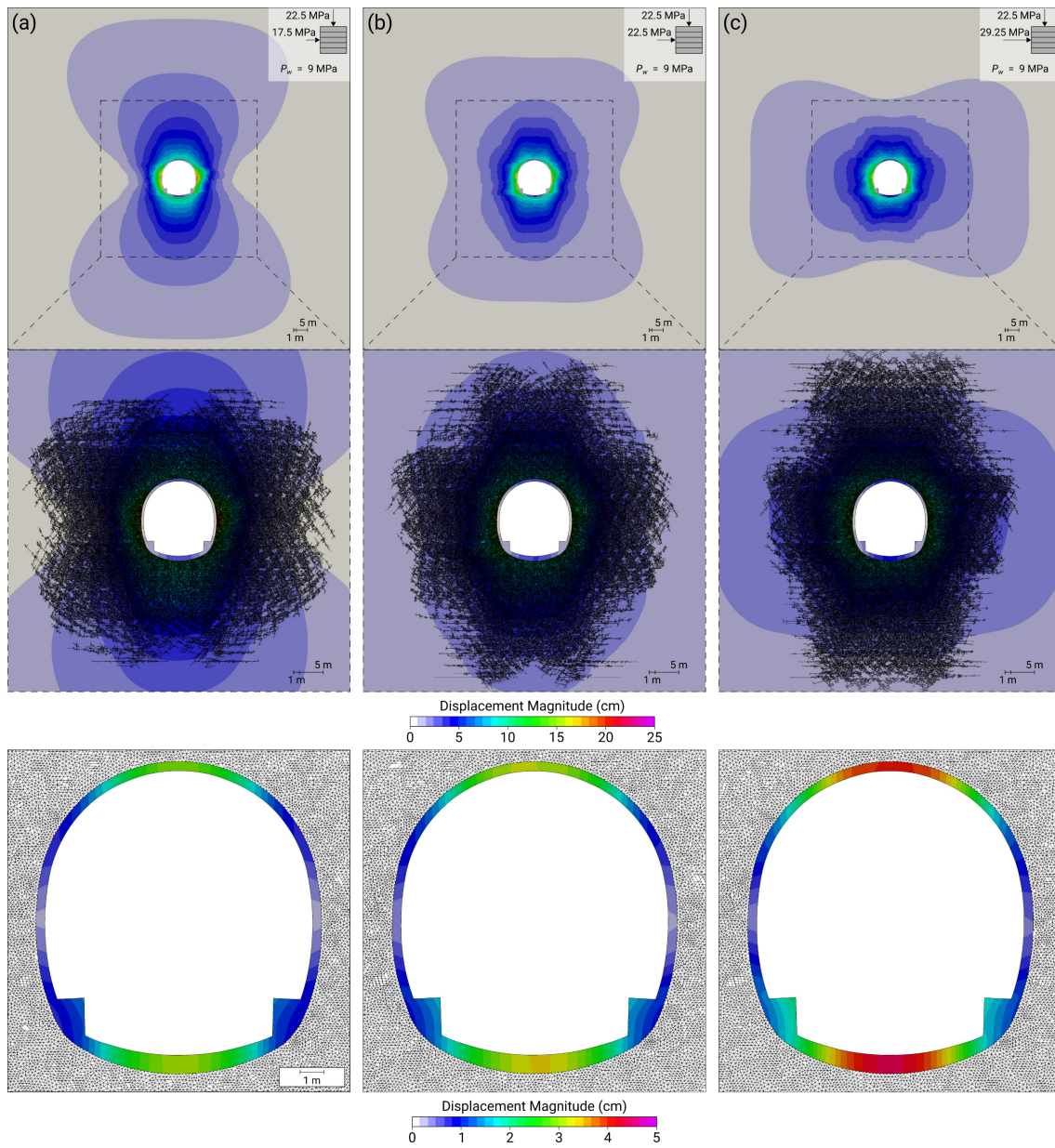


Fig. A-16: Displacement contours and associated EDZ fracture pattern for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. The liner displacement shown is the net displacement since the liner was installed. Positive displacement is towards the centre of the excavation

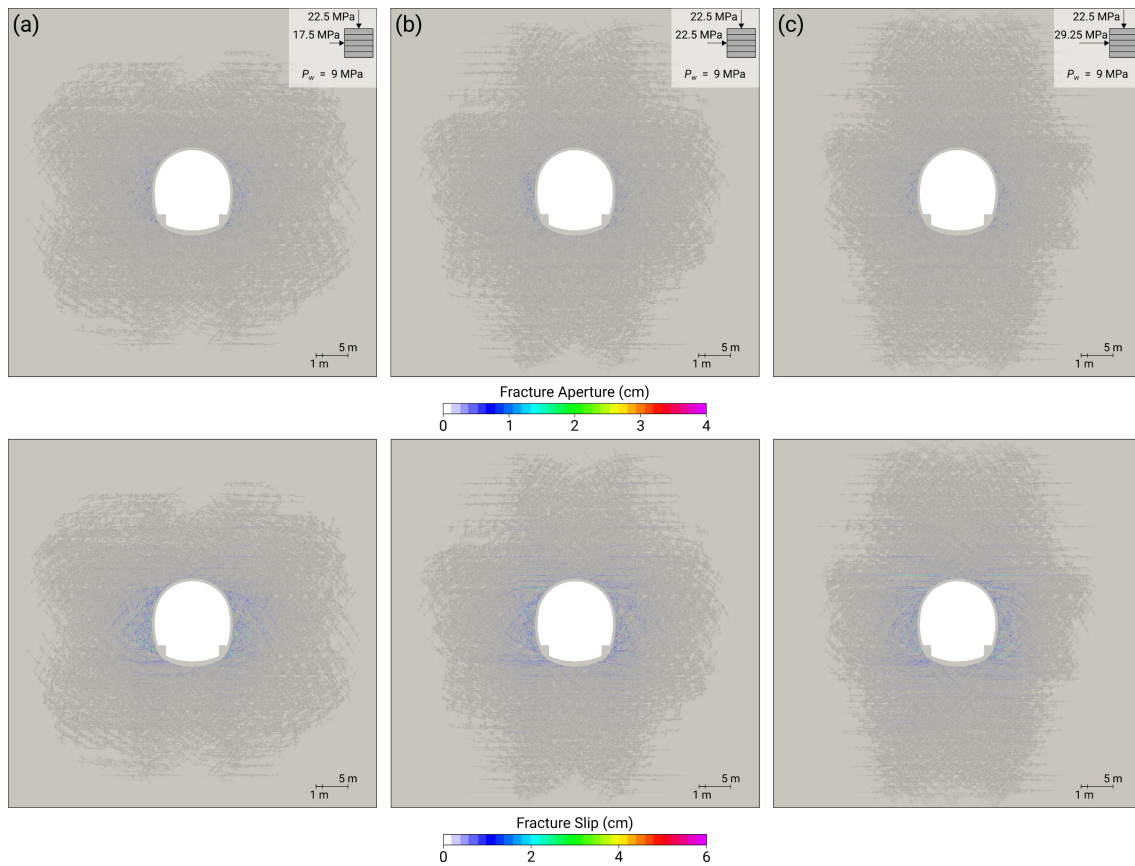


Fig. A-17: Fracture aperture (top) and slip (bottom) contours for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations

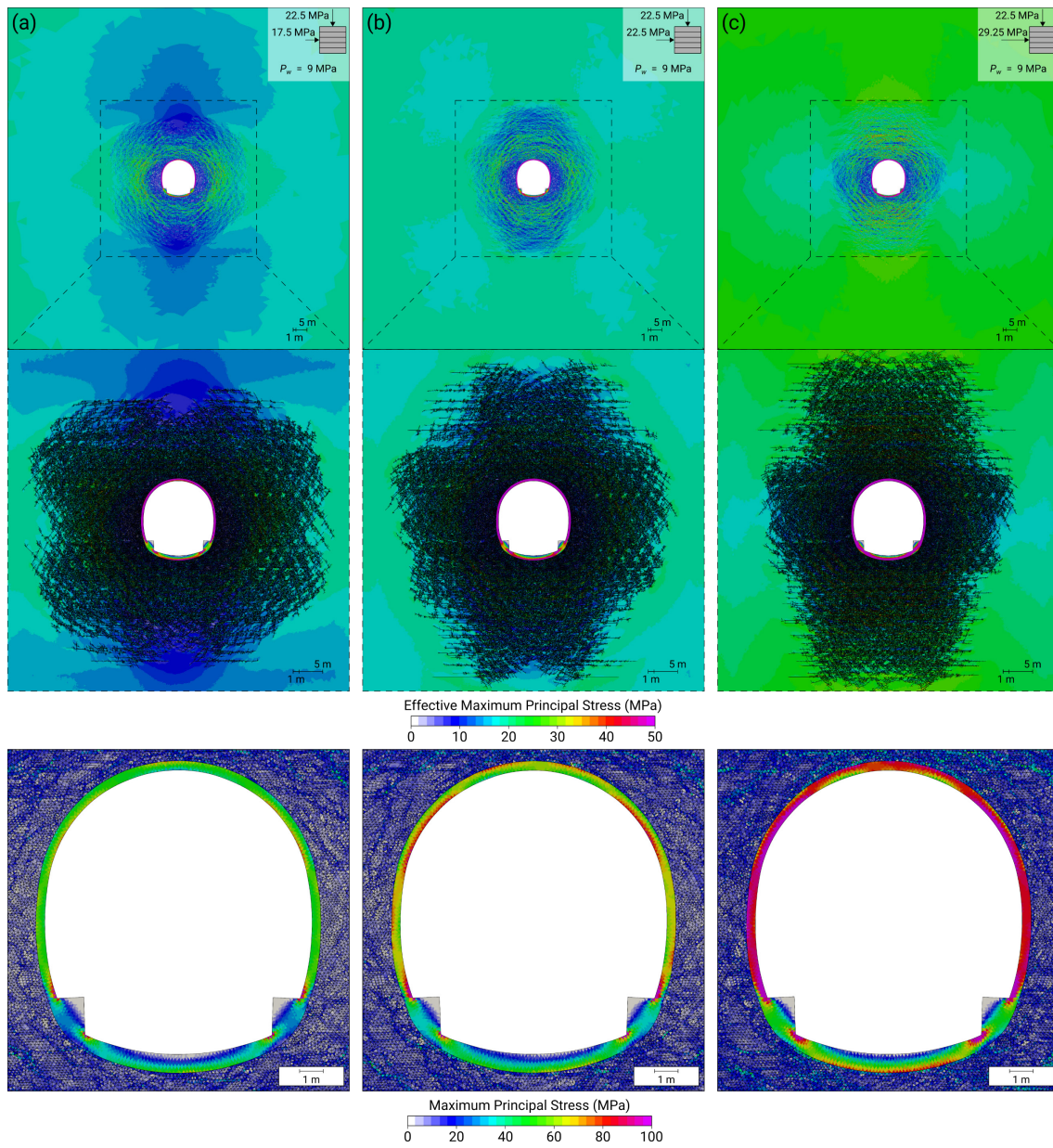


Fig. A-18: Effective maximum principal stress (top) and maximum principal stress (bottom) contours for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. Positive stress is compression

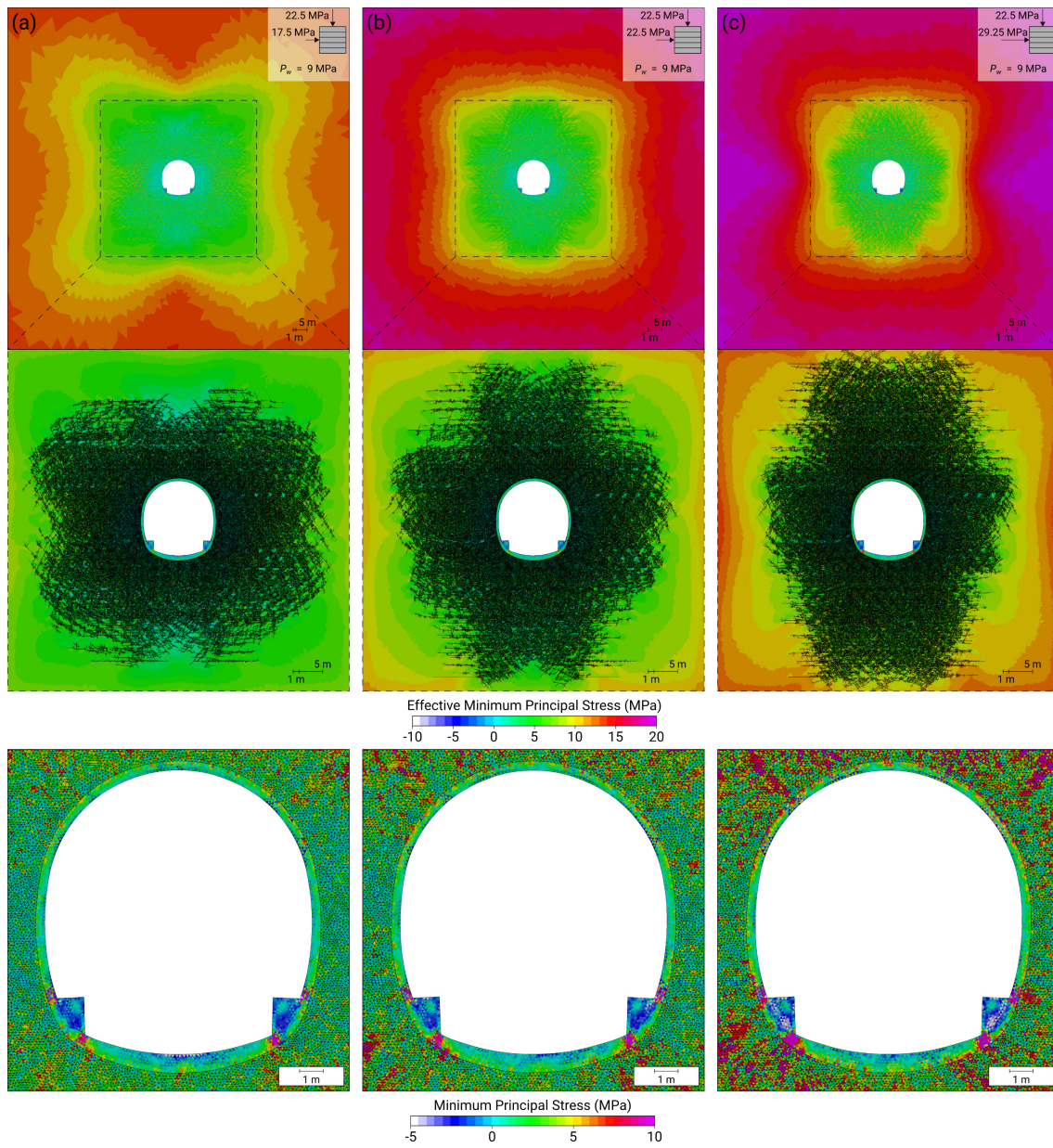


Fig. A-19: Effective minimum principal stress (top) and minimum principal stress (bottom) contours for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations. Positive stress is compression

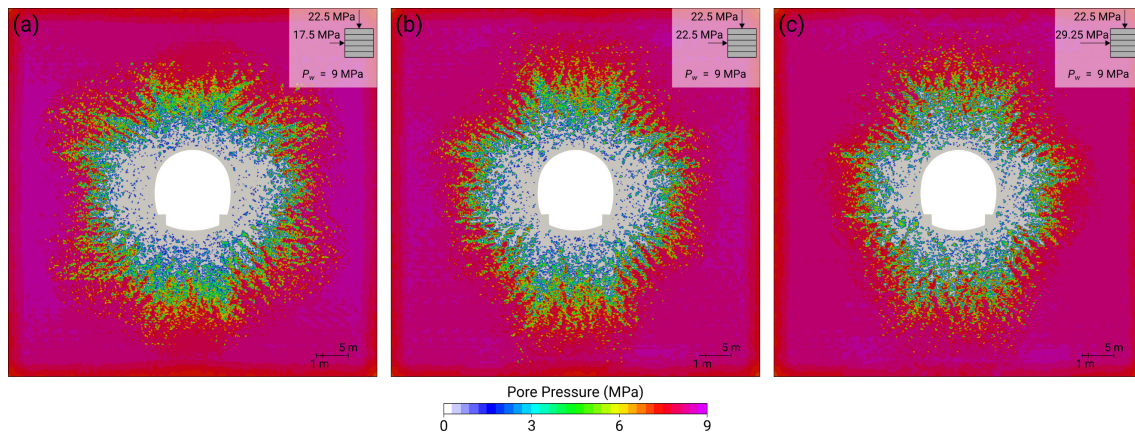


Fig. A-20: Pore pressure contours for L/ILW cavern models with (a) minimum horizontal in-situ stress ($\sigma_h = 17.5$ MPa), (b) isotropic in-situ stress ($\sigma_h = 22.5$ MPa), and (c) maximum horizontal in-situ stress ($\sigma_H = 29.25$ MPa) for lower bound strength calibrations.

App. B Core replacement technique and GRC concept

It is common engineering practice to carry out stress-strain analyses of tunnel cross-sections distant from the face under two-dimensional, plane strain conditions. However, the stress distribution in the rock mass surrounding an advancing excavation face is a three-dimensional problem. If a support system is installed behind the excavation face, part of the load re-distributed around the excavation is carried by the face itself. Consequently, the direct application of two-dimensional numerical models to the evaluation of the stress and strain fields in the cross-section of a supported tunnel may result in non-negligible inaccuracies (Curran et al., 2003). As the tunnel advances, the supporting action of the face decreases and, at a certain distance, plane strain conditions are attained. A two-dimensional analysis can be realistically employed only if the displacements of the excavation boundary prior to support installation can be estimated. Furthermore, in the context of this study, the amount of tunnel deformation was estimated from a finite element model (Madaschi et al., 2023) and applied to the model before the installation of lining.

To capture this effect, a core replacement technique pioneered by Curran et al. (2003) was implemented in this study. The three-dimensional face effect was captured by gradually reducing the radial resistance around the excavation surface using a fictitious, softening elastic material in the tunnel core. The Young's modulus of this material is gradually reduced from the original rock mass values, corresponding to an undeformed section far ahead of the tunnel face, to a value that results in the wall displacements required before liner installation.

Conceptually, the GRC is the deformation of the tunnel wall in response to changes in the radial pressure around the excavation (Fig. 3-7). In relation to the core replacement concept, the core reaction curve (CRC) is used to approximate the reduction in radial pressure as the excavation face advances. When the Young's modulus of the core is equal to the rock mass, there is no convergence of the excavation wall (i.e., an undeformed section far ahead of the tunnel face). As the Young's modulus of the core is reduced to approximate the three-dimensional excavation process, the radial pressure acting on the excavation wall is effectively reduced and in response the excavation converges. When the excavation converges to the amount of displacement expected at the time of liner installation, the core is completely removed (i.e., zero radial pressure) and the liner is installed. When installed, the rock mass will load the liner, deforming it according to the support characteristic curve (CRC) which is a function of the liner stiffness (dependent upon its geometry and mechanical properties). The intersection of the GRC and CRC is the point where the system is in equilibrium.