Numerical modeling of coupled thermo-mechanical response of a rock pillar

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Abstract: Understanding the rock mass response to excavation and thermal loading and improving the capability of the numerical models for simulating the progressive failure process of brittle rocks are important for safety assessment and optimization design of nuclear waste repositories. The international cooperative DECOVALEX-2011 project provides a platform for development, validation and comparison of numerical models, in which the Äspö pillar stability experiment (APSE) was selected as the modeling target for Task B. This paper presents the modeling results of Wuhan University (WHU) team for stages 1 and 2 of Task B by using a coupled thermo-mechanical model within the framework of continuum mechanics. The rock mass response to excavation is modeled with linear elastic, elastoplastic and brittle-plastic models, while the response to heating is modeled with a coupled thermo-elastic model. The capabilities and limitations of the model for representation of the thermo- mechanical responses of the rock pillar are discussed by comparing the modeling results with experimental observations. The results may provide a helpful reference for the stability and safety assessment of the hard granite host rock in China’s Beishan preselected area for high-level radioactive waste disposal.

Key words: thermo-mechanical coupling; Äspö pillar stability experiment (ASPE); numerical modeling; DECOVALEX-2011 project

1 Introduction

The coupled thermo-mechanical (TM) behaviors of surrounding host rocks have gained increasing research interest for safe disposal of radioactive waste. The excavation of the deep-buried underground openings and the heat generated by decay of radioactive wastes after their emplacements may induce damage and progressive failure of surrounding rocks. Understanding the rock mass response to excavation and thermal loading, therefore, is of paramount importance for safety assessment and optimization design of a nuclear waste repository. For this purpose, a large number of field experiments [1–6], laboratory tests [7–9] and numerical simulations [2, 6] have been performed in the world. Typical examples of field experiments include the mine-by experiment (MBE) and the subsequent heated failure tests (HFT) [1, 2] conducted at 420 m level in the Atomic Energy of Canada Limited’s Underground Research Laboratory (AECL’s URL) and the followed Äspö pillar stability experiment (APSE) [3–6] conducted at the 450 m level in the Äspö Hard Rock Laboratory (HRL), Sweden. The aims of these experiments are to investigate the excavation-induced and thermal loading-induced damages and progressive failure phenomena in crystalline hard host rocks under high deviatoric stress condition.

To assess the capability of the numerical models for simulating the complex processes involved in the progressive failure of the brittle rocks, and to provide predictive tools for future performance assessment of the radioactive waste repositories, comprehensive numerical modeling was performed for MBE and HFT with various continuum or discontinuum methods [2]. Numerical modeling of APSE has also been selected

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as the second task (Task B) of DECOVALEX-2011 (D-2011) project. The DECOVALEX project [10], short for “development of coupled thermo-hydro-mechanical (THM) models and their validation against experiments”, is an international cooperative research project on mathematical models of coupled THM processes for safety analysis of radioactive waste repositories. It has evolved since 1992 with five phases (D-I, D-II, D-III and D-THMC to now D-2011). The D-2011 project mainly includes the following three tasks:

(1) Task A: hydro-mechano-chemical (HMC) processes in argillaceous rocks and bentonite buffer involved in the Mont Terri ventilation test.

(2) Task B: coupled TM loading on hard rocks: pillar stability and fracturing near excavation rock surfaces involved in the APSE.

(3) Task C: fracture characterization and modeling, modeling of coupled thermo-hydro-mechano-chemical (THMC) processes in fractured granitic rocks, and flow and transport with sorption and diffusion in single fracture and fracture networks under normal and shear stresses.

According to the technical definition, Task B is divided into the following stages:

(1) Stage 1: to predict the rock mass response to excavation of the test tunnel and boreholes for forming the rock pillar. The numerical models are also required to be calibrated by an unconfined compressive strength (UCS) test and a triaxial test of core pieces.

(2) Stage 2: to predict the rock mass response to thermal loading by using elastic rock mass properties, and present the evolutions of temperature, stress and displacement of the rock pillar.

(3) Stage 3: to predict the rock mass response to excavation and thermal loading with discontinuous properties, present the progressive failure of the rock pillar, and compare the modelling results with experimental observations.

Two more stages, Stages 4 and 5, are under development at present, for follow-up researches on APSE.

Teams from Sweden (SKB), Finland (POSIVA), Czech Republic (TUL), UK (NDA), France (ISRN), Japan (JAEA), Korea (KAERI) and China (CAS and WHU) collaboratively participate in the research of Task B, among which the team from China includes two sub-teams from Chinese Academy of Sciences (CAS) and Wuhan University (WHU), respectively. The tasks of Stages 1 and 2 have so far been completed by most of the teams, and research for Stage 3 is still in progress.

This paper presents the modeling results of WHU team for Stages 1 and 2, Task B of D-2011 by using a coupled TM model within the framework of continuum mechanics. The capabilities and limitations of the model for description of the TM responses of the rock pillar are discussed by comparing the modeling results with experimental observations. The results may provide a helpful reference for the stability and safety assessment of the hard granite host rock in China’s Beishan preselected area for high-level radioactive waste disposal.

2 The Äspö pillar stability experiment

The APSE was conducted between 2002 and 2006 by the Swedish Nuclear Fuel and Waste Management Company (SKB) at 450 m level in Äspö HRL for testing the Swedish reference concept of deep geological storage of spent nuclear fuel in the crystalline Scandinavian shield [3–6]. The main objective of the experiment was to investigate the progressive failure process in a heterogeneous and fractured rock mass when subjected to coupled excavation-induced and thermal-induced stresses.

A test tunnel of 80 m in length was excavated using the drill and blast technique. The tunnel trends N46°E in the Äspö 96 coordinate system, which is approximately perpendicular to the direction of the major (horizontal) principal stress. The cross-section of the tunnel is made up of an arched roof, straight side walls and a rounded anti-arched floor, with a total height of approximately 7.5 m and a width of 5 m, as shown in Fig.1. A uniform concentration of excavation-induced stress in the floor was expected by the alignment and the shape of the tunnel.

The surrounding rock of the tunnel mainly consists of medium-grained unaltered Äspö diorite. Fractures are sparsely developed in the surrounding rock, mainly with three fracture sets; shear zones are also developed but less pronounced in the experimental volume, as shown in Fig.1(a). The quality of the sparsely fractured rock is characterized as “good” in the rock mass rating (RMR). The major and minor principal stresses in the surrounding rock are horizontal and trend N310°W and S220°W, with magnitudes of 30 and 10 MPa, respectively. The intermediate principal stress is vertical, with a magnitude of 15 MPa.
The experimental area is located between the chainages 59 and 69 m of the APSE tunnel. A rock pillar of 1.03 m in width was formed by excavating two vertical deposition holes, DQ0063G01 and DQ0066G01, of 1.75 m in diameter and 2.78 m in central distance by using a modified TBM, as shown in Fig.1. The designed depth of the boreholes was 6.5 m, but due to the technical difficulty involved at the end of the boring, the instrumented hole, DQ0063G01, was bored to 6.2 m.

The experiment mainly contains the following major steps:

1. Excavation and geological characterization of the APSE tunnel, and schematic design of the APSE.
2. Excavation of the confined hole, DQ0066G01, and installation of the confining system constructed of a rubber bladder. The average confining pressure was about 0.73 MPa in the experiment.
3. Excavation of the instrumented hole, DQ0063G01.
4. Installation of the monitoring system and heating system, as shown in Fig.2. The monitoring system consisted of 28 thermocouples, 23 short-range LVDTs, and 18 long-range LVDTs, etc. The heating system consisted of 4 electrical heaters with a designed heated length of 6.5 m. But the effective heated length of the heater in KQ0065G03 was reduced to 5 m due to presence of a steeply dipping water-bearing fracture (i.e. Fracture-08) intersecting the borehole at a depth of approximately 6.1 m.
(5) Heating of the pillar for 66 days from May 14 (identified as Day 0) to July 19, 2004, with the heating process shown in Table 1 [4], and then gradual releasing of confining pressure during July 14–15, 2004.

Table 1 Heating power per meter of heaters [4].

<table>
<thead>
<tr>
<th>Time (day)</th>
<th>KQ0064G04</th>
<th>KQ0065G02</th>
<th>KQ0064G05</th>
<th>KQ0065G03</th>
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<td>0</td>
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<td>46</td>
<td>263</td>
<td>263</td>
<td>400</td>
<td>520</td>
</tr>
<tr>
<td>66</td>
<td>0</td>
<td>0</td>
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</table>

(6) Slot drilling on the left side of the pillar after cooling for de-stressing, and excavation of the pillar by wire sawing into five blocks for further geological characterization and study of failure pattern inside the pillar.

The experimental measurements and observations show that the response of the pillar to excavation generally remained elastic, but was influenced by local variation of geological condition, a failed zone in forms of tensile fracturing and spalling occurred on the wall of hole DQ0063G01 at depths between 0.5 and 1.95 m, as shown in Fig.3. During heating, a V-shaped notch was formed close to the center of the pillar and propagated gradually from the excavation-induced failure zone down to 5 m in depth of the borehole. The depth of the notch was 20–130 mm, and the total spalled volume was about 0.1 m³. In the heating phase, however, the rock mass response was still considered to be elastic except in the failed zone [4].

The choice of a coupled thermal elastic or elastoplastic model for the rock pillar during the heating process is motivated by the above experimental observations that the rock mass response to excavation and heating largely remains elastic and by the modeling experiences for the MBE and HFT that an elastic model is capable of capturing the overall response of the rock mass in the experiments. A damage model for enhancing modeling progressive spalling of the rock by excavation and heating is under development for Stages 3–5 of Task B.

3 A coupled thermo-mechanical model

3.1 Governing equations

Within the framework of continuum mechanics [11, 12], the momentum conservation equations of a medium subjected to coupled TM loading in a quasi-static process can be written in the following rate form:

\[ \nabla \sigma + \rho g = 0 \tag{1} \]

where \( \sigma \) is the Cauchy stress tensor (positive for tensile stress), \( \rho \) is the density of the medium, and \( g \) is the gravitational acceleration vector.

The energy conservation equation of the medium can be expressed by

\[ \rho \dot{e} = q + \nabla \cdot \sigma \epsilon \tag{2} \]

where \( \dot{e} \) is the specific internal energy, \( q \) is the heat conduction flux vector, and \( \epsilon \) is the strain tensor.

The second law of thermodynamics is given by

\[ s \leq \left( \nabla \cdot \frac{q}{T} \right) \tag{3} \]

where \( s \) is the specific entropy, and \( T \) is the absolute temperature.

Supposing that the medium exhibits an elastoplastic behavior and the total strain, \( \dot{\epsilon} \), can be divided into an elastic part \( \dot{\epsilon}^e \) and an irreversible part \( \dot{\epsilon}^p \):

\[ \dot{\epsilon} = \dot{\epsilon}^e + \dot{\epsilon}^p \tag{4} \]

Defining the specific Helmholtz free energy by

\[ \psi = \psi (\epsilon - \epsilon^p, T) = e - sT \tag{5} \]

and substituting Eqs.(2) and (5) into Eq.(3) yield the Clausius-Duhem inequality related to a deformable medium in small deformation:

\[ \phi = \phi_m + \phi_h \geq 0 \tag{6} \]

where

\[ \phi_m = \sigma : \dot{\epsilon} - \rho s \dot{T} - \rho \dot{\psi} \geq 0 \tag{7} \]

\[ \phi_h = -\frac{q}{T} \nabla T \geq 0 \tag{8} \]

where \( \phi \), \( \phi_m \), and \( \phi_h \) are the total, mechanical and thermal dissipations, respectively.

![Fig.3 Failed zones on the pillar wall of DQ0063G01 after excavation and heating [4].](image-url)
Assuming that the thermal expansion of the medium exhibits an isotropic behavior, the state equations from Eq.(7) yield

\[ \sigma = \rho \frac{\partial \phi}{\partial \varepsilon} \quad \text{or} \quad \sigma = C (: (\dot{\varepsilon} - \varepsilon^p) - C : \alpha \delta \dot{T} ) \]  

\[ s = - \frac{\partial q}{\partial T} \quad \text{or} \quad s = \alpha \delta : C (: (\dot{\varepsilon} - \varepsilon^p) + c \dot{T} ) \]  

where \( C \) is the fourth-order tangential modulus tensor, \( \delta \) is the Kronecker delta tensor, \( \alpha \) is the linear thermal expansion coefficient, and \( c \) is the specific heat capacity of the medium.

Substituting Eqs.(9) and (10) into Eq.(7) finally yields the mechanical dissipation:

\[ \phi_m = \sigma : \varepsilon^p \]  

The entropy conservation can then be derived from Eqs.(2), (5) and (7):

\[ \rho T s = - \nabla q + \phi_m \]  

Substituting Eqs.(10) and (11) into Eq.(12) yields the following thermal equation:

\[ \rho c \dot{T} = - \nabla q + (\sigma : \varepsilon^p - \alpha T \delta : C : \varepsilon^p) \]  

The Fourier’s law for heat conduction is

\[ q = - \lambda \nabla T \]  

where \( \lambda \) is the second-order thermal conductivity tensor.

When a non-associated flow rule is adopted, the yielding and the irreversible strains of the material are determined by a yield function \( F \) and a plastic potential \( Q \), respectively, and \( \varepsilon^p \) can be written as

\[ \varepsilon^p = \zeta \frac{\partial Q}{\partial \sigma} \]  

where \( \zeta \) is a non-negative ratio that can be determined by the consistency condition of the yield criterion.

It is to be noted that, by setting \( \dot{\varepsilon}^p = 0 \), Eq.(15) immediately reduces to a coupled thermo-elastic model.

3.2 Thermal conductivity

It is clear from Section 3.1 that the coupled TM processes in a continuum are governed by Eqs.(1) and (13), supported by constitutive models presented by Eqs.(9), (14) and (15). Except for tangential modulus tensor \( C \), the parameters that should be indentified for the coupled TM model include thermal conductivity \( \lambda \), specific heat \( c \) and thermal expansion coefficient \( \alpha \). All of them are affected by temperature, density, mineral constituents, porosity, water content, and the shape and distribution of the microcracks of the rock concerned [13–16]. Due to a low porosity of the intact Åspö diorite, i.e. 0.2%–0.4% according to Staub et al. [3], the specific heat and the thermal expansion coefficient can be approximately taken as constants, but special concern should be paid to thermal conductivity.

There are large differences among the thermal conductivities of the solid, liquid and gas phases. As a result, the effective thermal conductivity of the rock mass is more affected by the above factors. Generally, the thermal conductivity of a rock mass may decrease and lead to anisotropy with the increase in porosity, decrease in water content and accumulation of damage. In a coupled model without consideration of the coupling effects of multiphase flow processes and damage evolution process, the impacts of porosity, water content and damage on thermal conductivity can not be fully represented. It can only be partly described as a function of temperature and stress.

In general, the thermal conductivity of a rock mass decreases with the increase in temperature [13, 14], and increases with the increase in pressure or axial stress [14, 15]. The latter is only valid for rock masses with elastic response or in some special directions, in which the increase in stress results in the closure of microcracks. By combining the research results presented in Refs.[13–15], we propose the following empirical expression to represent the dependence of thermal conductivity on temperature and stress [17]:

\[ \lambda = \frac{\lambda_0 + a \overline{\sigma}}{1.007 + (0.0036 - 0.0072 / \overline{\lambda_0}) T} \]  

where \( \lambda \) is the thermal conductivity (W·m\(^{-1}\)·°C\(^{-1}\)), \( \lambda_0 \) is the initial thermal conductivity in ambient temperature and stress-free conditions; \( \overline{\sigma} \) is the mean compressive stress (MPa); and \( a, b \) are coefficients. Görgülü et al. [15] listed a large set of values of \( a \) and \( b \) for various intact rock specimens in uniaxial compressive conditions, and concluded that these two coefficients may be correlated to elastic moduli of rock specimens. It should be pointed out that Görgülü’s expression can not be directly applied to the Åspö rock mass because the sparse fractures developed in the rock mass significantly reduce the thermal conductivity, and in Görgülü’s formulation, the mean compressive stress \( \overline{\sigma} \) in Eq.(16) is replaced by axial stress.

Calibration study shows that the effective thermal conductivity of 3.2 W·m\(^{-1}\)·°C\(^{-1}\) [4] is well suited for describing the thermal behavior of the Åspö diorite. Furthermore, if the initial thermal conductivity \( \lambda_0 \) is taken as 2.6 W·m\(^{-1}\)·°C\(^{-1}\) (a laboratory value [3]), and the parameters \( a \) and \( b \) are set to be 0.4 and 0.12, respectively, Eq.(16) is a good choice for the thermal conductivity of the Åspö diorite, as plotted in Fig.4, which clearly shows the dependence of thermal conductivity.
Fig. 4 Dependence of thermal conductivity on temperature and mean compressive stress for the Äspö diorite.

4 Numerical modeling for APSE

4.1 The finite element model

A cubic volume of 60 m×60 m×60 m around the APSE pillar was taken as the target area for Task B modeling. A finite element mesh was generated, with 147 320 brick elements and 155 705 nodal points. A finer mesh of 43 056 brick elements representing the tunnel and deposition holes to be excavated was used. The element size at the pillar boundary is 0.010 1 m (radial) × 0.114 3 m (tangential) × 0.31 m (high) to guarantee that the Gaussian quadrature points of the brick elements are approximately located 3 mm into the pillar wall in radial direction where the stress path is particularly concerned in Task B of D-2011, as shown in Fig.1(c).

In the present analysis, the surrounding rock was assumed as a homogeneous isotropic continuum. According to Refs.[3, 4], the material properties for the Äspö diorite are shown in Table 2. The computer code THYME 3D, initially developed for coupled deformation/multiphase flow/thermal transport analyses [18, 19], was used to perform simulations by disabling the multiphase flow and porosity evolution processes. The simulations of the APSE include the following two phases:

(1) Excavation of the test tunnel and deposition holes under isothermal condition. According to the technical definition of Task B, nine excavation stages were specified for excavation modeling. Three constitutive models were used and their calculation results were compared: a linear elastic model, an elastic-perfectly plastic model using the middle circle of Drucker-Prager (D-P) yield criterion and an elasto-brittle-perfectly plastic model using D-P yield criterion for both peak and residual strengths formulated by Zheng et al. [20, 21].

(2) Heating of the rock pillar for 66 days and then cooling for 34 days (100 days in total). In this phase, only the coupled thermo-elastic behaviors were modeled, considering the implications of the modeling results achieved in the first phase.

The normal displacements at all outer boundaries are set as roller boundaries of zero normal displacement. The initial stress field presented in Section 2 was used for excavation modeling, and the calculated stress field with the linear elastic model after excavation was taken as the initial stress field for the heating process of modeling. Considering the fact that the LVDT sensors were installed after excavation of the APSE tunnel and the large boreholes, however, the excavation-induced displacements were not accumulated for the heating process of modeling.

The initial temperature condition for the calculations was 15 °C [3]. The boundary condition for the thermal transport process was a constant temperature of 15 °C specified on the outer boundaries of the surrounding rock. For the excavation surfaces of the tunnel and deposition holes, calibration studies showed that if air convection and cooling effect of water leaked through the rock wall during the experiment were not considered, the temperature distribution would be highly overestimated even if the air in the tunnel and the deposition holes was regarded as a conductive medium. Therefore, the average temperature measurements by the temperature sensors in each deposition hole were prescribed on the surfaces of corresponding deposition hole. Since the
tunnel floor around the pillar was insulated with rock wool [4], the tunnel surface was assumed to be adiabatic in the simulation.

Heaters were modeled as linear heat sources. The positions of the heaters are illustrated in Fig.2. The input heating power of the heaters during the experiment is listed in Table 1 [4], and replotted in Fig.5. It should be noted that the power of heater in KQ0065G03 was corrected according to its effective heated length. In order to compensate the energy loss due to contact between heaters and rock, a reduction coefficient of 0.92 was taken for heaters in KQ0064G04 and KQ0065G02; while a smaller reduction coefficient of 0.60 was adopted for heaters in KQ0064G05 and KQ0065G03 to compensate the contact energy loss and the heat convection by water flow in the open water bearing Fracture-08. The above values of reduction coefficient were determined by fitting the measured temperature curves at the monitoring sensors installed in KQ0064G06, KQ0064G07 and KQ0064G08 (see Fig.2).

4.2 Modeling results for excavation

The predicted maximum tangential stress at point A (see Fig.2), located at 1.95 m in depth and 0.003 m into the pillar wall at the narrowest part of the pillar in DQ0063G01 for the excavation stages, is plotted in Fig.6 (positive for compressive stress hereafter). It should be noted that the prediction results of stress path at point A are highly affected by mesh size and stress smoothing technique. To guarantee precision of the stress calculation results, as mentioned previously, the size of finite element mesh is controlled so that point A is located at Gaussian quadrature points of some brick elements in radial direction (see Fig.1(c)). The elastic and the elastoplastic models yielded very similar results, while the brittle-plastic model predicted significantly lower stresses at the last three steps of excavation due to abrupt drop in rock strength from peak to residual as soon as the peak strength was reached.

The predicted stress paths at point A by using the three different constitutive models are plotted in Fig.7. Also the stress path predicted using the boundary element code Examine 3D with elastic properties [4] is plotted, as marked by symbol SKB in Fig.7. Again, the calculation results show that the above three constitutive models behave similarly during the excavation of the first hole, while the brittle-plastic model differs from the other two during the excavation of the second one. Besides, the modeling results with linear elastic or elastoplastic models are rather close to SKB’s data. During excavation of the tunnel and hole DQ0066G01, the principal stresses and mean stress at point A increase, and the excavation-induced disturbance and damage at point A are insignificant. During excavation of hole DQ0063G01, however, \( \sigma_1 \) at point A decreases, \( \sigma_2 \) increases and \( \sigma_3 \) dramatically decreases.
increases, leading to an increase in mean stress and a drastic increase in deviatoric stress. After the excavation is completed, the $\sigma_1$ value at point $A$ approaches the lower bound of the UCS of the Åspö diorite, and exceeds the crack initiation stress. Such a stress state may result in brittle failure and spalling of the weak area on the rock wall (see Fig.3).

As excavation is completed, the accumulated deformation responses of the surrounding rock predicted by the elastic model are shown in Figs.8 and 9, with the maximum displacements on the tunnel walls. According to the calculation, the horizontal convergence of the tunnel at the section of chainage 49 m is 6.8 mm, which is very close to the measured results (6.5 mm) [3].

An interesting phenomenon about the deformation pattern of the surrounding rock during excavation is that the directions of the displacement vectors are not all pointing towards the excavated space. In the centre of the pillar between the two deposition holes (at the cross-section perpendicular to the tunnel), there exhibits a contracted deformation towards the floor of the holes, due to the magnitudes and orientations of the initial stresses in the field. From this result, one may assume that the release of $\sigma_1$ during the formation of the heating-induced yield notch in the pillar should be an important reason for the radial expansion or pillar contraction recorded by the LVDTs at the centre instrument positions at depths of 3.0 and 3.5 m in the initial stage of heating [4].

The distributions of the principal stresses of the surrounding rock first predicted by the elastic model and the plastic zone predicted by the elastoplastic model after excavation of the holes are shown in Figs.10 and 11, respectively. One may observe that by simply using an elastic model or an elastoplastic model with D-P yield criterion, the failure process of the pillar (see Fig.3) can not be accurately captured. For this purpose, the local geological characteristics of the rock mass should be considered and more sophisticated constitutive models or discontinuum methods should be used. Taking into account the limitations of the continuum models, therefore, only the coupled thermal elastic model is used to model heating in the next phase.
4.3 Modeling results for heating

4.3.1 Temperature distribution

The distribution of temperature at the horizontal section of 3.5 m in depth on Day 65 is depicted in Fig.12. The results of temperature evolutions at various instrumentation locations during heating are plotted in Figs.13–15. It should be noted that the predicted temperature for the two deposition holes is trivial since the wall surfaces of the holes are set as thermal boundaries with specified temperature, which is taken to be the average of measured temperature in each hole.

In hole KQ0064G06 at various depths, the predicted trend of temperature evolutions is in good agreement with the measurements, with small to moderate discrepancies in magnitude, as shown in Fig.13. The measurements show the highest temperature curve at 1.5 m in depth and the lowest curve at 3.5 m in depth, with the temperature curve at 5.5 m in depth between them. The predictions, however, show the highest temperature curve at 3.5 m in depth and the lowest temperature curve at 5.5 m in depth, with the curve at 1.5 m in the middle. After switch-off of heating system, the predictions agree well with the measurements.

In hole KQ0064G07 at various depths, the predicted temperatures agree well with the measurements at depths of 1.5 and 5.5 m, but the predictions are larger...
than the measurements at 3.5 m in depth. Such a discrepancy can also be observed after switch-off of the heaters, as shown in Fig.14. In hole KQ0064G08 at various depths, the predicted temperatures are larger than the measurements. Especially at the depth of 2.75 m, the difference is up to 6 °C, as shown in Fig.15.

The main reasons that lead to the above discrepancies in magnitude between the predicted and measured temperatures are: (1) simplifications of geometry, geology and boundary conditions; (2) negligence of the effects of groundwater flow, the water flow in the water-bearing fractures, and the air convection in the tunnel and hole openings on thermal transport; and (3) negligence of the dependences of thermal conductivity on saturation, fractures, damages and microstructures. According to Task B definition, these issues will be addressed in Stages 3–5.

4.3.2 Thermal stress

The thermal stresses at the points 0.003 m into the pillar wall at the narrowest part at depths of 2.5 and 4.1 m due to the heating predicted by the coupled thermo-elastic model are plotted in Fig.16. It can be observed that at 2.5 m in depth, the maximum tangential stress increases from 114.4 to 155.2 MPa, while at 4.1 m in depth, it increases from 99.8 to 139.6 MPa. Actually, at these points, the tangential stresses are approximately equal to the maximum principal stresses. The heating has a negligible influence on the minimum principal stress, with an increase of no more than 0.33 MPa; but has a moderate impact on the evolution of the intermediate principal stress, with an increase of 10.1–12.3 MPa. As a result, the deviatoric stress significantly increases, leading to the failure and spalling of the pillar wall. From Fig.16, it can be inferred that the localized tensile failure may occur first in some weak zones of the pillar and then propagate downwards to the bottom, since the stress concentration at the upper part of the pillar wall is much larger than that at the lower part.

The principal stresses along the pillar, 3 mm into the wall at the narrowest part of the pillar, on Days 0, 35 and 60 are plotted in Fig.17. One observes that with the increase in the depths from 0 to 6 m, the maximum principal stress decreases drastically from 193.66 to 66.96 MPa on Day 0. As the heating proceeds, the maximum principal stress increases significantly, with the stress distribution curve moving towards the right side. The stress increase reaches the maximum, i.e. 42.07 MPa, from Day 0 to Day 60 at 3.1 m in depth. On the other hand, the change in the minimum principal stress with the depth during the heating stage is negligible at depths between 0.5 and 5.0 m. Its distribution at the depth below 5.0 m fluctuates, which may be most probably due to stress concentration at the bottom of the hole and the shape of finite elements.

![Fig.16](image1.png)  
**Fig.16** Evolutions of predicted maximum tangential stresses at points of 3 mm into the pillar wall at depths of 2.5 and 4.1 m.

![Fig.17](image2.png)  
**Fig.17** Distributions of principal stresses along the pillar 3 mm into the wall at the narrowest part on Days 0, 35 and 60.

The above calculations show that the intact hard rock at the top of the pillar seems to bear a deviatoric stress up to 196 MPa. The stress may significantly decrease if the excavation disturbed zone (EDZ) is considered. But the predictions correctly show the trend that the tensile failure may start at some weak parts of the pillar wall and then propagate downwards.
as a result of the thermal stress and the localized stress concentration around the notch.

4.3.3 Heating-induced deformation

The radial displacements at the narrowest positions of the pillar wall at depths of 2.5 and 4.1 m during heating are plotted in Fig.18. The distributions of the radial displacement along the pillar at the narrowest position at various times are depicted in Fig.19. The curves show that pillar expansion (i.e. radial contraction of the deposition holes) occurs during heating, and the maximum elastic radial displacement at the narrowest position only reaches 0.21 mm after 60 days of heating. The value is smaller than the LVDT measurements, but is still large enough to yield large deviatoric stress and trigger tensile failure and spalling of the pillar wall for the intact hard Åspö diorite. It is the tensile fracture that causes the drastic increase in radial deformation of the pillar wall.

![Displacement vs. Time](image1.png)

**Fig.18** Evolutions of radial displacements at the narrowest positions of the pillar at depths of 2.5 and 4.1 m.

![Displacement vs. Depth](image2.png)

**Fig.19** Distributions of radial displacements along the pillar at the narrowest position at depths from 0 to 6 m during heating.

The predicted evolutions of displacements at the short range LVDT positions at depths of 2.5 and 4.1 m are plotted in Fig.20. Again, the predicted maximum displacement only reaches 0.39 mm, a value far smaller than the maximum LVDT measurements after spalling occurred.

![Displacement vs. Time](image3.png)

**Fig.20** Evolutions of displacements at the short range LVDT positions at depths of 2.5 and 4.1 m.

It is obvious that without a mechanical mechanism to model the progressive failure process of the hard rock, the coupled thermo-elastic model fails to capture the irreversible deformation and failure of the rock, as recorded by the short range LVDTs. Other reasons that may account for the discrepancies between the measured and predicted displacements include the validity of assumed homogeneity and isotropy of the surrounding rock and the lack of description of brittle damage in the EDZ.

5 Conclusions and discussions

The paper presents the WHU team’s efforts on developing a coupled TM model and its calibration against the APSE. In the current stage, the evolutions of stress and deformation of the pillar during excavation are modeled by using an elastic model, an elastoplastic model and a brittle-plastic model. The heating-induced temperature, thermal stress and deformation evolutions are modeled by a coupled thermo-elastic model. The modeling results agree well in trends with measured data, but with varying discrepancies in magnitudes, as expected by most of the simplified thermo-elastic and thermo-plastic modeling approaches requested at Stage 2 of Task B. More comprehensive modeling methods considering progressive developments of damage and failure (spalling) processes are needed.

Numerical modeling for excavation of the tunnel and the deposition holes shows that the mechanical responses predicted by the elastic and the elastoplastic models behave almost the same, while the brittle-plastic model differs from the other two during the excavation of the second hole, due to the abrupt jump of rock strength from peak to residual as
long as the peak strength is reached. The calculation results show that both the elastic and the elastoplastic models are not able to accurately describe the progressive failure process of the pillar during excavation, and detailed geological characterizations and more sophisticated constitutive models or discontinuum methods should be used for assessment.

The coupled thermo-elastic model predicts that heating of the pillar may result in an increase in tangential stress on the pillar wall up to 42 MPa and hence a drastic increase in deviatoric stress. Therefore, tensile failure may start at some weak parts of the pillar wall and then propagates downwards as a result of the thermal stress and the localized stress concentration around the notch.

The predicted maximum radial displacement during heating is in the magnitude of 0.4 mm, which is much smaller than the measurements after spalling occurs. The irreversible deformation due to spalling cannot be modeled by the coupled thermo-elastic model. Other limitations include the assumption of homogeneity and isotropy of the surrounding rock and the lack of description of brittle damage and tensile failure mechanisms in the EDZ.

The predicted temperatures evolve in a similar trend with the measurements, but discrepancy occurs in magnitude. The main reasons include: (1) simplifications of geometry, geology and boundary conditions; (2) negligence of the effects of underground water flow, the water flow in the water-bearing fractures, and the air convection in the tunnel and hole openings on thermal transport; and (3) negligence of the dependences of thermal conductivity on saturation, fractures, damages and microstructures.

Overcoming the above limitations of the model will be the major task of the cooperative research for the remaining stages (Stages 3–5) of Task B of the DECOVALEX-2011 project.

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