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Assessment of site suitability for lined rock caverns in repurposed mine drifts for underground hydrogen storage

L. Bäcklund & P. Zhang
Luleå University of Technology, Luleå, Sweden
lena.backlund@ltu.se
0009-0006-4939-5439
0000-0002-9278-1283

Abstract

Lined rock caverns (LRCs) are an underground gas storage solution which offers safe and high-capacity gas storage at flexible locations, without the geological limitations associated with other storage methods. With the increasing global focus on reducing greenhouse gas emissions, underground hydrogen storage combined with LRC has been proposed as a solution for seasonal energy storage. However, one of the main limitations is the high cost of LRCs since the gas tightness is engineered by using concrete and steel liners rather than based on the geological properties of the site, meaning higher construction and material costs. Repurposing underground drifts by using LRC technology in inactive mining areas has been suggested as a potential alternative.

This paper investigates the feasibility of constructing lined rock drifts (LRDs) in inactive mining areas of a Swedish underground mine. The conceptual study was carried out through numerical simulations based on published material available about the site. Pre-existing excavations of varying dimensions and depths were analyzed. Assuming a constant maximum gas pressure and fixed minimum thicknesses of concrete and liner, the effects of liner stress and the overall rock conditions were studied to aid in site selection. The results indicated that the rock quality and the depth are the most important aspects to consider, as when they both increase, the adverse effects of a fully pressurized LRD are significantly lessened. The size of the drift was found to be less impactful for the mechanical response but should be taken into account when considering the material usage and cost of the construction.

Keywords

Lined Rock Cavern, Lined Rock Drift, Repurposing of mine infrastructure, Numerical modelling.

1 Introduction

Large-scale hydrogen storage is expected to play an important role in future low-carbon energy systems by balancing variable renewable power and industrial hydrogen demand. Underground storage is particularly attractive where large volumes are required, but the suitability of different storage concepts depends strongly on local geological conditions. In Sweden, where salt caverns and depleted hydrocarbon reservoirs are limited, lined rock caverns (LRCs) have become a relevant alternative for large-scale underground hydrogen storage.

A LRC is an engineered structure, consisting of an innermost gastight liner, a concrete liner to transfer the load, a drainage layer, and finally the rock mass which is the main load-bearing component. The LRC concept itself is well established through earlier Swedish development work, including pilot testing at Grängesberg and successful natural-gas storage demonstration at Skallen. The technology has also been proven suitable to store gaseous hydrogen with the HYBRIT pilot project (Johansson, 2003; Johansson et al., 2018; Swedish Energy Agency, 2024). One of the main drawbacks of the technology is its high cost. As a result, efforts are being made to minimize excavation expenses during construction, for example by optimizing design approaches and repurposing existing underground infrastructure (Luo et al., 2024; 2025). However, Studies that specifically examine the reuse of mine infrastructure are still limited and have mainly focused on compressed air energy storage in closed or abandoned mines (Schmidt et al., 2024; Xu et al., 2021). Hydrogen-specific applications in repurposed mine openings are less commonly explored, with recent work by Liu et al. (2024) providing one of the clearest direct examples of converting an abandoned mine into an LRC for hydrogen storage.

This paper investigates the conceptual feasibility of repurposing legacy excavations at the Kiirunavaara mine for hydrogen storage, specifically in its mine drifts. Using numerical modelling to perform a parametric study, the influence of depth, rock mass quality and drift size on liner demand is assessed to identify potentially favourable underground locations for further study.

2 Mine site description

2.1 Kiirunavaara mine

Kiirunavaara mine, located in Kiruna in northern Sweden, is an iron ore mine which has been in continuous production since 1898. The deposit has historically been mined by open pit methods near surface, but since the 1960s extraction has been carried out underground by sublevel caving, which remains the dominant mining method today (LKAB, 2023).

2.2 Mine-wide infrastructure

The mine-wide underground infrastructure at Kiirunavaara has progressively developed as production has advancedto greater depths. A sequence of main levels was developed to facilitate the transport of ore to the surface. Earlier main levels were located at depths of 275, 320, 420 and 540 m, followed by 775 m, 1045 m and, most recently, 1365 m. All main levels are situated on the footwall side of the orebody, where significantly less deformation and fracturing occur compared to the hangingwall side. On these main levels there are former haulage drifts, railway-based transport levels, shafts and associated service openings from previous mining stages (Johansson and Öberg, 1996; LKAB 2023).

2.3 Geology and rock mass

The Kiirunavaara orebody is tabular and semicontinuous, approximately 4 km long, striking roughly north-south and dipping 55-60° eastward. The ore is located at the contact between trachyandesitic volcanic rocks of the Hopukka formation in the footwall and rhyodacitic to dacitic–rhyolitic volcanic rocks of the Luossavaara formation in the hanging wall (Bergman, 2018). Its thickness varies between 0-200 m, but averages around 80 m.The footwall rocks are mainly intermediate volcanic to subvolcanic rocks, locally grading into syenitic rocks and commonly affected by alteration and veining (Jacobsson et al., 2013; Björnell et al., 2015). In general, the footwall rock is of high quality, although it exhibits considerable variability, with uniaxial compressive strength varying from 90 to 430 MPa (Sjöberg et al., 2012; Malmgren, 2001).

3 Numerical model description

3.1 Model geometry

The modelling software used to conduct the parametric study was RS2, a 2D continuum FEM program. The lined rock drifts (LRDs) were analysed assuming plane strain conditions. The outer boundaries of the cases are illustrated in Fig. 1. The geometry shown in a) was adopted for most models, while the geometry in b) was used to assess the effect of a shallow overburden.

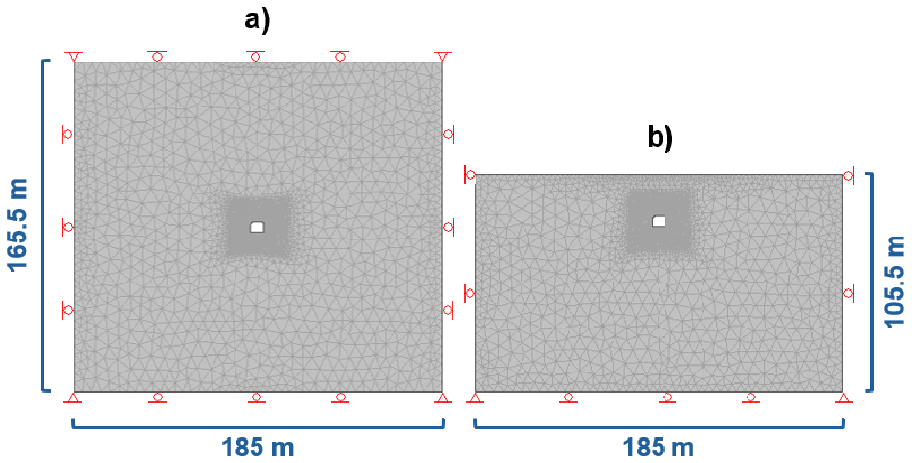


Fig. 1 Outer boundaries of the 2D models analyzed in this paper: a) at depth, and b) the shallowest 20m case.

The model geometry after lined rock drift installation is shown in Fig.2, with each component being marked in numbers. To simplify the study, the shotcrete with included drainage layer was omitted, with a singular concrete liner representing both this layer and the concrete lining. The internal steel liner and sliding layer were modelled as a liner support element and an underlying joint between the steel and concrete liners, respectively.

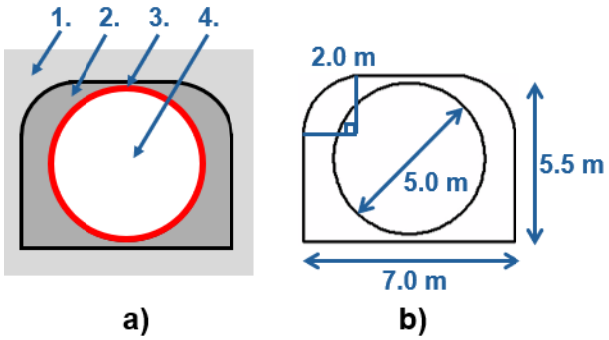


Fig. 2 Conceptual illustration of the lined rock drift and its dimension. a) Lined rock drift components:1. Rock, 2. Concrete,3. Sliding layer + Steel liner, 4. Gas cavity; b) Geometry of the drift and its dimension.

3.2 In-situ stress and boundary conditions

The in-situ stresses used were based on the work done by Sandström (2003), and the relationship is shown below in Eq. 1:

$$\begin{aligned} \sigma_{ew} &= 0.037z \\ \sigma_{ns} &= 0.028z \\ \sigma_{v} &= 0.029z \end{aligned} \tag{1}$$

Where σ_{ew} is the in-situ stress in the east-west direction (MPa), σ_{ns} is the stress in the north-south direction (MPa), σ_v is the vertical stress (MPa) and the z is the depth (m). Given that the drift was oriented in the east–west direction, the corresponding east–west stress was applied in the out-of-plane direction, whereas the vertical stress acted along the y-direction, and the north-south stress along the x-direction in the numerical model.

The boundary conditions used for the models depended on the depth being evaluated. Displacement was constrained in the direction normal for all cases except those at shallow depth. For drifts at depths of 20 m and 80 m, the top boundary was left free to allow movement.

3.3 Parameters

The rock mass parameters were based on the footwall base case as presented in Sjöberg et al. (2012) and the structural material properties were based on the ones used by Luo et al. (2024), all of which have been compiled in **Error! Reference source not found.** below. The rock mass and concrete were assumed to behave perfectly plastically, while the steel liner was modelled elastically. The sliding layer was modelled as an interface between the liner and the concrete. The inner gas pressure for all cases was set as 20 MPa.

Table 1. Material properties used in the model.

Attribute	Unit	Rock mass	Concrete	Liner	Sliding layer
Density, ρ	[kg/m^3]	2800	2400	8000	-
Young's modulus of intact rock, E	[GPa]	70	30	200	-
Poisson's ratio, v	[-]	0.27	0.20	0.3	-
Intact material constant, m_i	[-]	20	-	-	-
Intact uniaxial compressive strength, σ_{ci}	[MPa]	300	-	-	-
Geological strength index, GSI	[-]	75	-	-	-
Friction angle, φ	[°]	-	45	-	-
Tensile strength, σ_T	[MPa]	-	2.7	355	0
Cohesion, c	[MPa]	-	9.6	-	0
Normal stiffness, K_N	[GPa/m]	-	-	-	45
Shear stiffness, K_S	[MPa/m]	-	-	-	43

3.4 Modelling procedure and studied cases

The modelling was conducted in the following stages: the model was first allowed to reach equilibrium pre-excavation, and then post-excavation. The lined rock components were then installed (Concrete liner, steel liner with sliding layer) and the model was allowed to reach equilibrium again with the LRC components being initially loaded with their own self weight. Finally, the LRC was pressurized, and the liner stress was calculated for the whole circumference using Eq.2:

$$\sigma_{min,max} = \frac{F}{t} \pm \frac{MC}{I} \quad (2)$$

Where $\sigma_{min,\ max}$ is the minimum and maximum stresses in a liner segment at a certain point with moment M and induced axial force F; t is the thickness of the liner; C denotes the centre-to-edge distance, with a maximum value of $t/2$, and I is the moment of inertia. The maximum tensile stress for each case was then recorded to provide a basis for comparison among the cases.

The study considered the following parameters to narrow down the area of interest. The base case marked with an asterisk (*) represented the default values used unless a parameter was varied.

1. **Depth:** A lined rock drift can potentially be placed at several different levels; therefore, defining an acceptable range was of interest. The overburden heights investigated were*20 m, 80 m, 200 m, 430 m*,* and *600 m.*
2. **Rock quality:** The rock quality in Kiirunavaara mine is in general good, however it has a variety of rock types with varying quality (Sjöberg et al., 2012).To investigate these effects, the geological strength index (GSI) values tested were:*45, 60, 75*,* and *90*.
3. **Drift geometry:** To evaluate the effect of drift size, the geometry was varied. For consistency in comparison, the drift dimensions were scaled proportionally, while the thicknesses of the steel and concrete liners were kept constant. The drift sizes investigated were: *4 m ×3.143 m, 7 m ×5.5 m*, 10 m ×7.057 m*, as shown side-by-side in Fig. 3 a)-c).

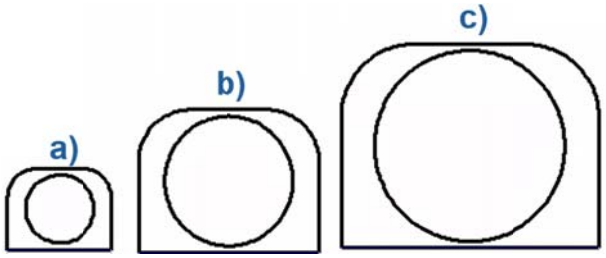


Fig. 3 The different drift geometries being investigated: a) 4 m ×3.143 m, b) 7 m ×5.5 m and c) 10 m ×7.057 m.

4 Results and site evaluation

The results were evaluated partly by looking at the yielding condition of the rock and concrete, as well as studying the maximum tensile stress in the steel liner and comparing it to its yield strength of 355 MPa. The axial force component of Eq. 2 remained consistent over the circumference of the steel liner, with an increase in the lower hemisphere, as the resultant force of gravity was aligned with the resultant force from the internal pressure. The moment component, however, varied more, peaking at the edges and across the yielded zones causing bending induced stress at these locations. So, with these two factors interacting, the maximum tensile stress determined by Eq. 2 was mainly located near or on a yielded zone in the concrete in the lower hemisphere of the steel liner.

4.1 Drift depth

The effect of the depth was clearly reflected in both the rock and the concrete, See **Error! Reference source not found.** below. At 20 m depth, the analysis showed extensive yielding in the concrete. In the surrounding rock, plastic zones developed diagonally to a distance up to 26 m, accompanied by a tensile yield zone developing from the surface along the drift's central axis. The corresponding surface displacement at ground level was approximately 2.5 mm. At 80 m depth, the extent of concrete yielding was reduced and the plastic zone in the rock was limited in extent, reaching only 2-4 m evenly around the excavation. The surface displacement for this case was negligible (<0.2 mm). For drift depths greater than 200 m, tensile yielding was confined to the concrete, with little or no observable yielding in the surrounding rock. It was also noted that within the 20-200 m depth range, the rock does not fail prior to LRC pressurization. Minor yielding near the excavation boundary was observed at greater depths of 430 m and 600 m. In these cases, post-excavation yielding is minimal, consisting of isolated elements extending approximately 0.1 m into the rock mass.

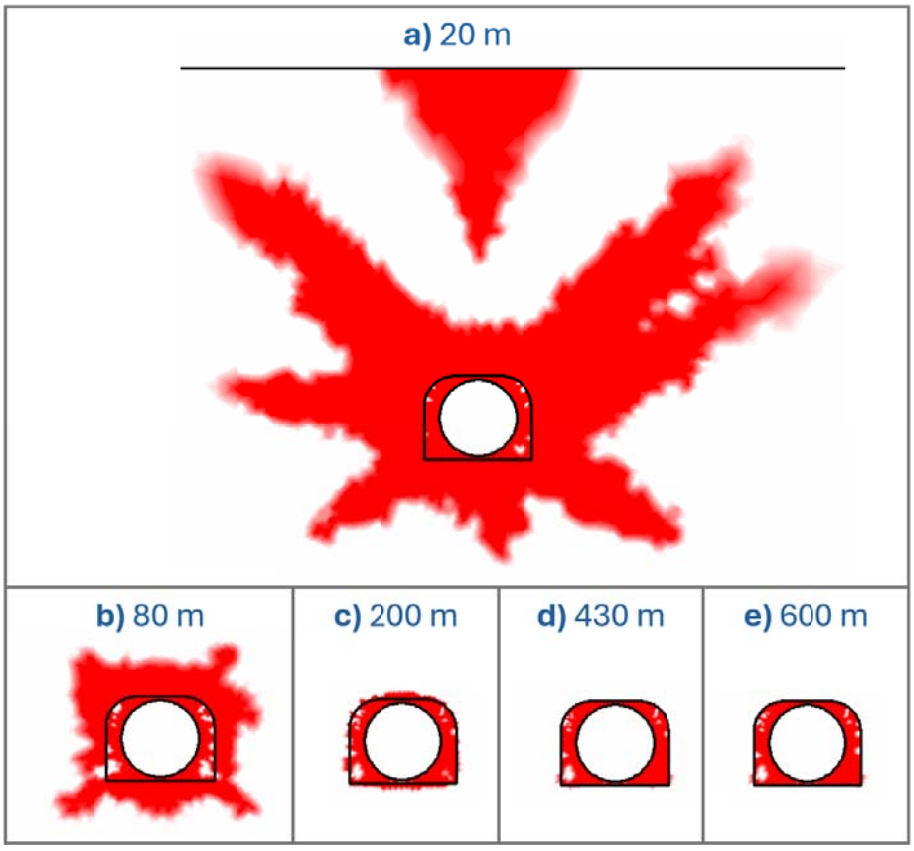


Fig. 4 Yield zone for the different depth cases. a) 20 m overburden, with ground surface included; b) 80 m overburden; c) 200 m overburden; d) 430 m overburden and e) 600 m overburden.

The liner stress at different depths is compiled in a diagram in **Error! Reference source not found.** below. At 20 m, the liner stress exceeded the yield strength of the steel liner, but beyond that, the influence of the overburden was not sufficient to affect the yield state of the liner with minimal change from 200 m onwards.

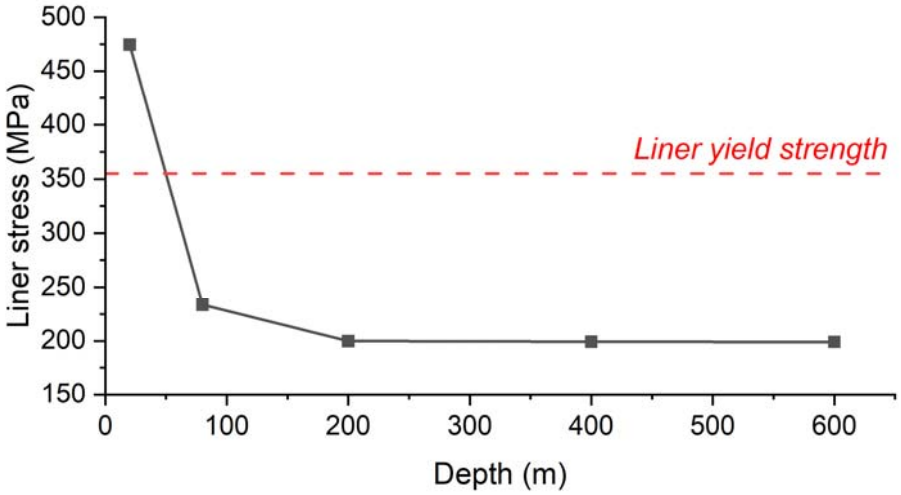


Fig. 5 Variation of liner stress at different drift depths. Compression negative.

4.2 Rock mass quality

Tensile yielding in the concrete occurred in all cases, however its extent varied. The concrete was fully yielded in both the GSI 45 and 60 cases, with the extent of yielding getting less widespread as the rock quality increased. For GSI 45, the surrounding rock yielded in shear, in a relatively uniform zone extending approximately 0.5 m from the excavation boundary, with the main post-pressurization yielding occurring in the drift floor. At GSI 60, the yielding in the rock was less uniform and was mainly concentrated at the drift floor and lower corners, extending about 0.3 m. For GSI values of 75 and above, the rock yielding was minimal.

The liner stress for the GSI parametric study is shown in Fig. 4. The primary effect was observed in the lining stress, where lower rock mass stiffness resulted in higher forces and bending moments, leading to increased stresses in the liner.

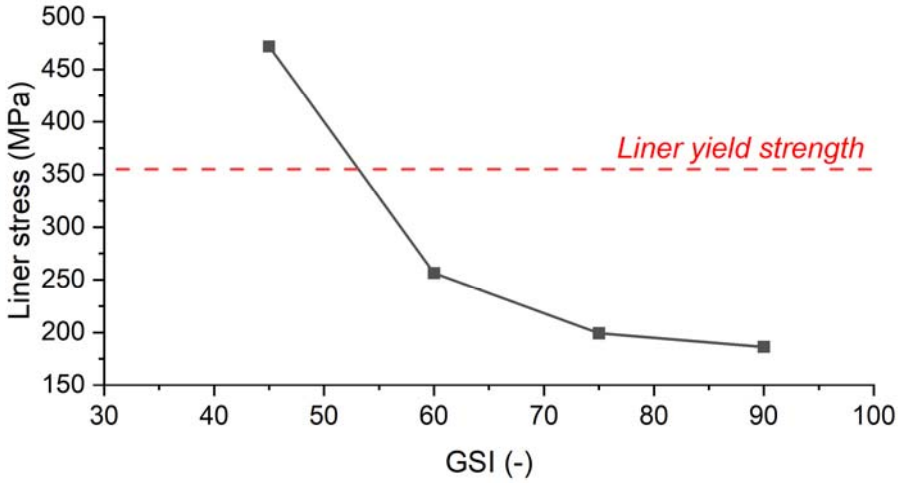


Fig. 4 Maximum tensile stress in the steel liner as a function of GSI. Compression negative.

4.3 Drift size

The results of changing the drift dimensions are presented in Fig. 5. Unlike the previously investigated parameters, the size of the drift had no significant effect on the liner stress, although a slight declining trend was observed with the drift size (and corresponding liner radius) increase. There was yielding in the concrete in all cases, which became progressively more widespread as the drift size increased.

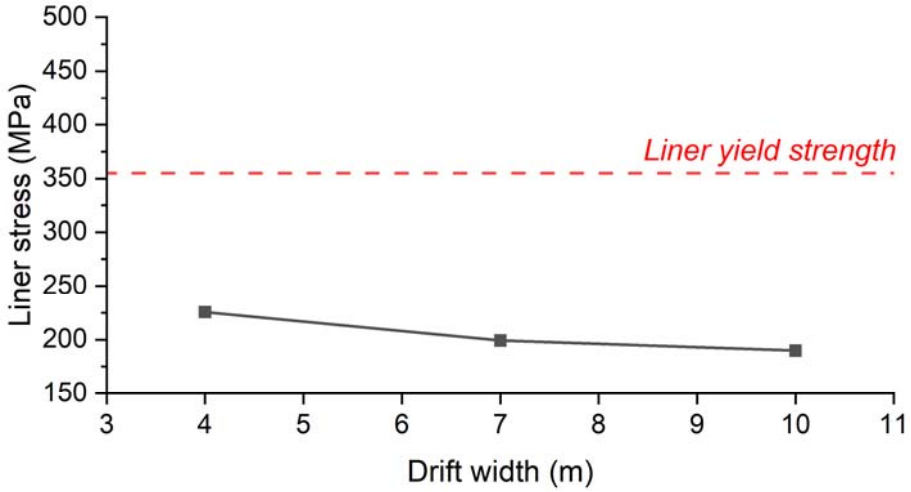


Fig. 5. Maximum tensile stress in the steel liner at different drift widths. Compression negative.

5 Discussion and conclusion

Overall, the results indicated that underground hydrogen storage in a mining setting represents a viable option. The adopted modelling approach revealed a clear point of diminishing returns beyond a depth of approximately 200 m, both regarding the rock and concrete yielding as well as the maximum tensile stress in the steel liner. Rock mass quality exhibited a more gradual effect on the performance of the LRD system; however, assuming that the GSI of 75 is reasonably representative, the mine drifts on the footwall side are likely to be suitable. The limited influence of drift size on liner stress is an advantage, as it implies that the material usage per unit of hydrogen stored is lessened. Nevertheless, it's worth mentioning that there will be an increase of post-excavation damage with the increase of excavation dimension. Consequently, depending on the rock quality and in-situ stress state, an optimal excavation dimension must be evaluated which was not addressed in this paper. The chosen dimensions in this paper were based on geometries that could be feasibly found in Kiirunavaara mine; pre-existing mine excavations larger than approximately 10 m × 7 m would likely require new excavations and are therefore beyond the scope of this paper.

An important limitation of the present study is the use of a continuum FEM approach, which does not explicitly capture discrete cracking in the concrete lining or the localized interaction between concrete cracking, joint opening, and strain concentrations in the sealing system. This is particularly relevant for lined rock caverns, as previous work has shown that crack development in the concrete lining can strongly influence liner response, and that the opening of rock joints may induce local strain concentrations in the lining (Johansson et al., 2018; Liu et al., 2025). The present results should therefore be interpreted as representing the global structural response rather than detailed local damage evolution. While some cracking and yielding in the concrete is unavoidable in LRCs, properly assessing the extent and severity of this behaviour is important before any definitive and final statement regarding suitability is made.

Several aspects should also be addressed in future work. Kiirunavaara mine is seismically active, and it must be ensured that any potential seismic events do not affect the stability and operability of LRC installations. In addition, the effects of the excavation damaged zone (EDZ) and any pre-existing damage in the surrounding rock were not considered in this study. This omission is particularly relevant in the context of repurposed drifts, where the excavation history may have created a heterogeneous and difficult-to-characterize damage zone. Kim et al. (2013) showed that the potential for tensile fractures in the concrete lining can be substantially reduced if the EDZ around the cavern is minimized. However, since the drifts considered here were excavated long ago, the current condition and extent of any EDZ remain unknown and should be investigated in future work.

6 Acknowledgements

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