On the short-term response of Opalinus Clay to tunnelling

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ABSTRACT: Opalinus Clay is the host rock for the Swiss deep radioactive waste repository. Effective stress analyses have been performed for a first assessment of hazards related to the short-term ground response to the excavation of the repository drifts and caverns. The analyses were performed assuming that the rock either remains saturated or completely desaturates under the negative pore pressures developing short-term around the tunnels due to cavity unloading. The results show that the risk of shield jamming during construction of the repository drifts can be mitigated by providing an over-cut in the range of just a few centimetres. Yielding support is planned for the construction of repository caverns in order to accommodate deformations and reduce rock pressure. An over-excavation of one to two decimetres, in combination with adequate structural detailing, must be planned to accommodate the short-term convergences.

Keywords: radioactive waste repositories, Opalinus Clay, brittle softening, squeezing ground, short-term, negative pore pressure cut-off.

1 INTRODUCTION

The Swiss National Cooperative for the Disposal of Radioactive Waste (Nagra) identified the Opalinus Clay formation as the most suitable for long-term waste containment. A so-called combined repository is planned, which will include 14 m diameter caverns for storage of low- and intermediate-level waste (L/ILW) and 3.5 m diameter drifts for storage of high-level waste (HLW) (Figure 1). The repository sites currently under consideration are located 600 to 900 m below the surface, within a 100 m thick saturated layer of Opalinus Clay. Squeezing conditions are expected when tunnelling through Opalinus Clay at such depths.

Shield tunnelling is foreseen for the construction of the HLW-drifts. Under the expected conditions, shield jamming is a potential construction hazard (see, *e.g.*, Ramoni & Anagnostou 2010). The L/ILW-caverns are to be excavated conventionally, full face, and with a yielding support to accommodate deformations and reduce rock pressure. Providing an adequate over-excavation is one of the crucial measures to avoid inadmissible deformations (see, *e.g.*, Cantieni & Anagnostou 2009a).

Opalinus Clay exhibits - due to its extremely low permeability (in the order of 10^{-13} m/s) - a pronouncedly time-dependent response to tunnel excavation. The time-dependency is associated with the very slow dissipation of the excavation-induced excess pore pressures. The short-term behaviour of Opalinus Clay is characterised by a constant water content, and is important for the interaction between rock, tunnelling equipment and tunnel support in the first months after excavation.

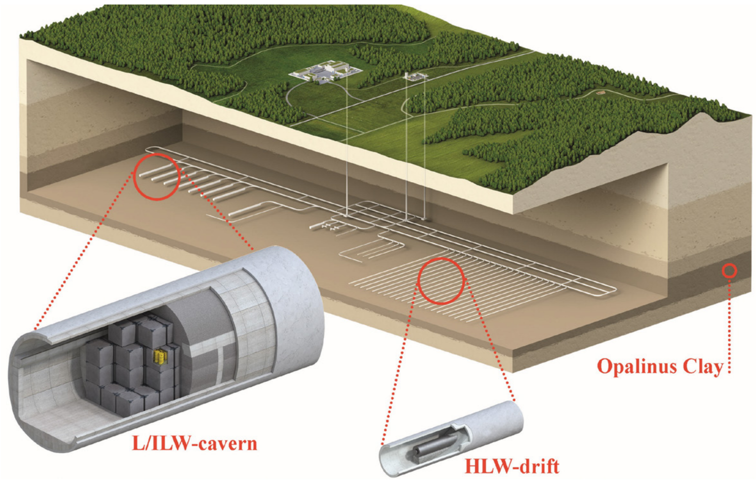

Figure 1. Schematic layout of a combined L/ILW- and HLW-repository.

2 OPALINUS CLAY

Opalinus Clay is a mostly grey to black, silty to sandy claystone. Its mechanical properties have been investigated by means of consolidated drained and consolidated undrained triaxial compression tests. The tests have been performed for various bedding orientations (see, *e.g*., Crisci *et al*. 2021). The typical experimental behaviour is characterized by: slightly non-linear stress-strain behaviour right from the start of loading; stiffness anisotropy; strength anisotropy (lower strength for failure along the bedding compared to failure through the matrix); practically brittle strain softening in the rock matrix and bedding; and moderate stiffness dependency on the initial confining pressure.

Typical material constants calibrated from the experimental results are given in Table 1. Stiffness anisotropy is represented by different Young's moduli orthogonal and parallel to the bedding. Mean elasticity parameters are taken for the (rotationally symmetric) ground response analyses. Strength anisotropy is considered by distinguishing between failure governed by rock matrix and by bedding. Both peak and residual strengths are considered. The dependency of stiffness on the confining pressure is considered by taking the Young's modulus equal to the one corresponding to the *in-situ* stress at repository depth. For the depths of 600 and 900 m, *in-situ* total stresses of 15 and 22.5 MPa and effective stresses of 9 and 13.5 MPa are considered.

Table 1. Elasticity and plasticity parameters of the Opalinus Clay (Anthi *et al*. 2022).

Elasticity parameters	**Depth**	**600 m**	**900 m**
Young's modulus orthogonal to bedding E_o	[MPa]	3'720	4'530
Young's modulus parallel to bedding E_p	[MPa]	11'160	13'590
Mean Young's modulus E_m	[MPa]	7'440	9'060
Poisson's ratio for stresses orthogonal and strains parallel to bedding ν_{op}	[-]	0.1	0.1
Poisson's ratio for stresses and strains parallel to bedding ν_{pp}	[-]	0.15	0.15
Mean Poisson's ratio ν_m	[-]	0.125	0.125
Plasticity Parameters		**Matrix**	**Bedding**
Peak cohesion c / Residual cohesion c_r	[MPa]	5 / 1	3 / 1
Peak angle of friction φ / Residual angle of friction φ_r	[°]	33 / 27.5	29 / 21.5
Peak angle of dilatancy ψ / Residual angle dilatancy ψ_r	[°]	10 / 4	5 / 2

3 SHORT-TERM GROUND RESPONSE CURVES

Analytical ground response curves (GRC) cannot account for anisotropic ground behaviour and non-hydrostatic initial stresses. Nevertheless, as we will see later, they are valuable for a first estimation of the ground response to tunnelling and the assessment of hazards during construction.

Figure 2 shows short-term (undrained) GRCs for openings in Opalinus Clay at depths of 600 and 900 m (l.h.s. and r.h.s. diagrams, respectively), considering either the matrix strength parameters (upper diagrams) or the bedding strength parameters (lower diagrams). Each diagram contains four curves: the blue curves assume that the rock remains 100% saturated even at high negative pore pressures (GRC calculation after, *e.g.*, Vogelhuber 2007), while the red curves consider a "suction cut-off", *i.e.* they assume that the rock completely desaturates as soon as the pore pressure drops below zero (GRC calculation after Nordas *et al*. 2023a). The solid lines hold for brittle-softening behaviour (characterized by a sudden drop from peak to residual strength), while the dashed lines have been obtained under the simplifying assumption of perfectly plastic behaviour with the residual strength parameters. Consequently, Figure 2 considers all possible combinations of extreme (borderline) cases, thus bounding the possible range of ground responses.

The diagrams point to the following conclusions: (i) If the rock remains saturated at negative pore pressures, then the displacements are very small (less than 1% of the tunnel radius). (ii) Suction cut-off (desaturation) results in larger displacements of up to 5%. (iii) For the ground parameters and *in-situ* stresses considered, the GRCs obtained under the simplifying assumption of perfectly plastic behaviour with the residual strength parameters are practically identical to those obtained for brittle softening (see also Nordas *et al*. 2023b).

Considering the uncertainties about rock behaviour under high negative pore pressures, the most unfavourable borderline case, that of suction cut-off in combination with residual strength parameters, is relevant for the design considerations outlined in the next sections.

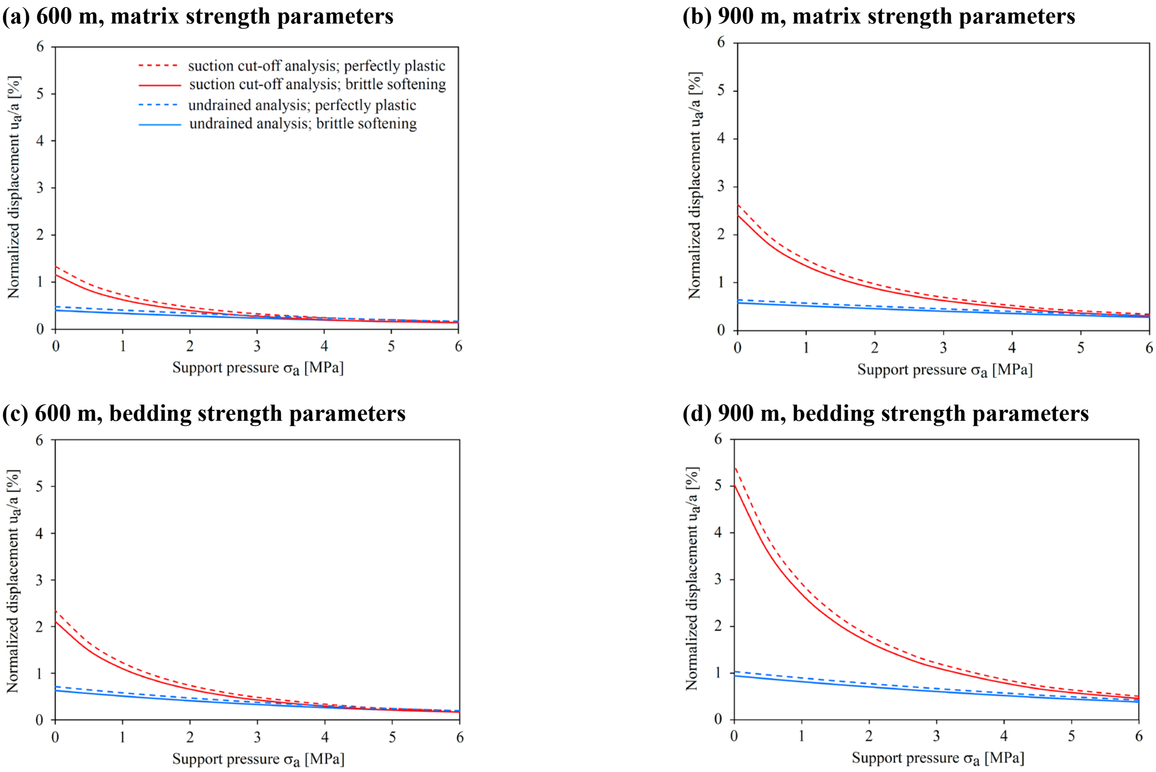

Figure 2. Ground response curves based on effective stresses.

Total stress analyses assuming perfectly plastic behaviour with the residual strength parameters show results very similar to those for effective stress analyses with suction cut-off (Figure 3). As noticed already by Boldini & Graziani (2012), the overall mechanical response of the plastic zone is, due to desaturation, similar to that of a dry medium. It can be thus assumed that the known methods of longitudinal displacement profile (LDP) estimation (which are based upon total stress analyses) are also approximately valid for ground response analyses that consider short-term conditions, effective stresses and suction cut-off, and can be used for estimating the ground displacements that occur before lining installation.

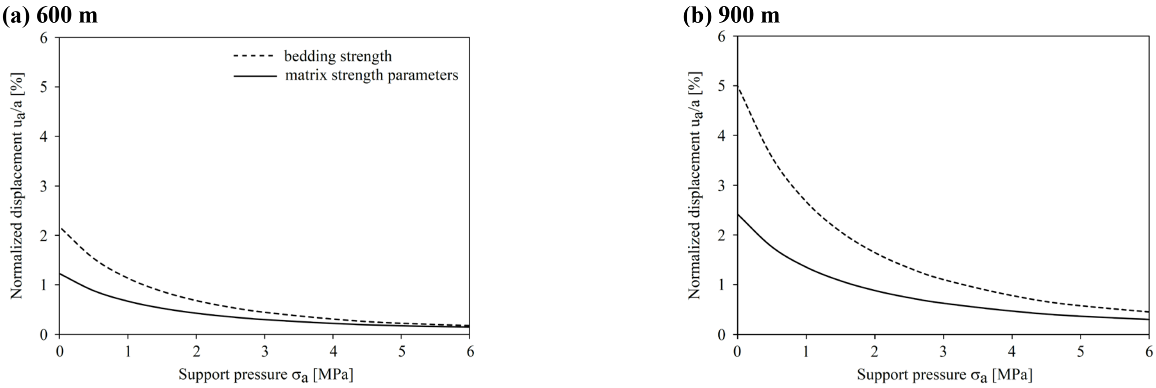

Figure 3. Ground response curves based upon total stresses and considering perfectly plastic behaviour with the residual strength parameters.

4 ANALYSIS FOR THE HLW-DRIFTS

Due to the small diameter of the HLW-drifts, the shield will be relatively long (ca. 7 m, that is twice the diameter). The gap between the ground and the segmental lining will be grouted immediately behind the shield tail. The potential hazard of shield jamming can be mitigated by providing an overcut sufficiently big to avoid contact between rock and shield. Table 2 estimates the necessary overcut based upon the GRCs presented above and the so-called implicit method of LDP estimation.

Table 2. Estimation of required radial gap to mitigate jamming of the TBM shield.

Depth of cover		600 m		900 m	
Plasticity parameters		Matrix	Bedding	Matrix	Bedding
Normalized total ground displ. u_a/a ($\sigma_a = 0$) [(1)]	[-]	1.2%	2.1%	2.4%	5.0%
Total ground displacement ($\sigma_a = 0$)	[mm]	21	37	42	88
Portion of total displ. 7 m behind the face [(2)]	[-]	69%	51%	55%	32%
Ground displacement 7 m behind the face	[mm]	14	19	23	28
Ground displacement at the face [(3)]	[mm]	4	5	6	7
Ground displ. between face and shield tail	[mm]	10	**14**	17	**21**

(1) from Figure 2 brittle plastic behaviour with suction cut-off considering a diameter of 3.5 m (a = 1.75 m)
(2) after implicit method (Guo 1995; Nguyen-Minh & Guo 1996) for supported opening (30 cm lining with E = 20 GPa)
(3) taken equal to 25% of the displacement that occurs 7 m behind the face

According to Table 2, an overcut of just 14 mm (600 m depth) or 21 mm (900 m depth) would suffice to avoid ground - shield interaction and thus jamming of the shield, even when considering - in the sense of a conservative simplifying assumption - the lowest conceivable strength parameters (those of the bedding). Such overcuts are quite feasible and can easily be achieved through an appropriate TBM design. However, it is well known that analyses of the interaction between ground and support based on ground response curves may be inaccurate for the case of stiff supports installed close to the face (Cantieni & Anagnostou 2009b). Numerical computations considering spatial effects are thus required for in-depth analysis of the problem. Such analyses are presented by Nordas *et al.* (2023b) based on a preliminary parameter set; they consider full hydromechanical coupling, strength, stiffness and in-situ stress anisotropy as well as TBM advance in 3D, and result in higher, but still manageable convergences (27-40 mm for 900 m depth). Respective analyses with the same parameter set as presented in Table 1 result in a required overcut of *ca.* 24 mm for 900 m depth (Natale *et al.* 2022). This agrees very well with the results in Table 2.

5 ANALYSIS FOR THE L/ILW-CAVERNS

Analogously to the analysis for the HLW-drifts, emphasis is placed here on the necessary deformation margin of the yielding support. It is assumed that the yielding support exerts a pressure of 0.5 MPa during the deformation phase. Such a pressure is relatively high for a 14 m diameter opening, but can be materialized employing special high-strength ductile concrete elements. The required over-excavation is estimated analogously to last section, based on the short-term ground response curves of Figure 2 (Table 3). The case considering desaturation is also decisive for those computations. According to Table 3, the yielding support must be able to accommodate a radial displacement of up to 63 mm (600 m depth) or 151 mm (900 m depth) in the short-term. This can easily be achieved through appropriate structural detailing.

Table 3. Assessment of required radial over-excavation of the yielding support to accommodate short-term displacements during construction of the L/ILW-cavern.

		600 m		900 m	
Depth of cover		Matrix	Bedding	Matrix	Bedding
Normalized total ground displ. u_a/a (σ_a = 0.5 MPa) [(1)]	[-]	0.8%	1.5%	1.8%	3.6%
Total ground displacements (σ_a = 0.5 MPa)	[mm]	56	105	126	252
Portion at ca. 4-5 m behind the face [(2)]	[-]	40%	40%	40%	40%
Displ. to be accommodated by the support	[mm]	34	**63**	76	**151**

(1) from Figure 2 brittle plastic behaviour with suction cut-off considering a diameter of 14 m (a = 7 m)
(2) considering development of convergence for the case of elasto-plastic ground for unsupported opening after Corbetta (1990)

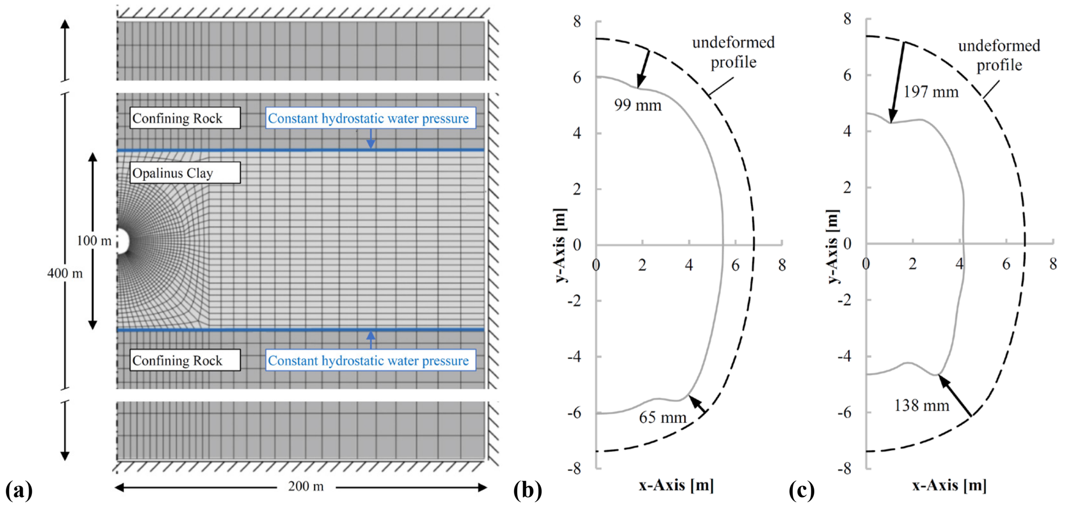

Figure 4. Numerical model (a) and short-term deformation profiles at a depth of 600 m (b) or 900 m (c).

The analytical, rotationally symmetric analyses cannot account for anisotropic material behaviour. In order to consider the strength- or stiffness-anisotropy of Opalinus Clay, plane strain FEM computations have been performed, assuming linearly elastic - perfectly plastic stress-strain behaviour with the material constants of Table 1, whereby only residual strength parameters have been considered for reasons explained in Section 3. Figure 4 shows the numerical model and the short-term ground displacements for the depths of 600 and 900 m. The numerical computations consider the actual profile of the L/ILW-cavern and confining rock of higher permeability. The numerically obtained total short-term ground displacements for a support pressure of 0.5 MPa agree well with the range computed with the ground response curves: 65-99 mm (numerically for 600 m, Fig. 4b) vs. 56 to 105 mm (analytically, Table 3); 138-197 mm (numerically for 900 m, Fig. 4c) vs. 126 to 252 mm (analytically, Table 3).

6 CONCLUSIONS

For the conditions expected at the repository sites the short-term response of the Opalinus Clay to tunnel excavation is relevant for the assessment of TBM shield jamming during the construction of the HLW-drifts and for the estimation of the required over-excavation of the yielding support of the L/ILW-caverns. Short-term GRCs are very valuable for a first assessment of hazards during construction in very low permeable rock such as the Opalinus Clay. The results of GRC-based analytical computations agree remarkably well with those of cumbersome and time-demanding 2D or 3D numerical analyses. GRCs considering ground desaturation are decisive for both cases as the assumption of suction cut-off results in higher short-term displacements than the case without desaturation. For the project-specific conditions and parameters, the response of a material exhibiting brittle softening and suction cut-off is very close to that of a perfectly plastic material with residual strength. Further it was shown that the short-term response with suction cut-off is very close to that of a dry ground.

The first assessments show that shield jamming is not critical even at a depth of 900 m and even under the conservative assumption of residual bedding strength in all directions. It was further shown that the required over-excavation for the construction of the caverns is within feasible ranges.

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