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Key design issues of lined tunnels and shafts used for compressed air energy storage

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Planning and Designing Tunnels and Underground Structures

Compressed air energy storage (CAES) in a lined rock cavern (LRC) taking the form of a tunnel or shaft represents an alternative to pumped-storage reservoirs for storing large quantities of energy. The internal gas pressure is borne by the rock, while the tightness of the system is guaranteed by a thin steel lining. The present paper discusses some important hazard scenarios for LRC tunnels or shafts at relatively shallow depths in several different geotechnical conditions. We show that the safety of the rock mass with respect to uplift failure is a necessary but not a sufficient condition for assessing feasibility. Where the rock is relatively soft, buckling and fatigue failures of the steel lining are more important when evaluating the feasible operating air pressure. The stability of the concrete plug that seals the compressed air stored in the cavity must also be investigated. We show, however, that the strength of the rock mass is not decisive in this respect.

Keywords: Compressed air energy storage, tunnels, shafts, lining, plug.

1. Introduction

Energy storage is of growing importance due the increasing exploitation of renewable energy sources of an intermittent nature. Compressed air energy storage (CAES) plants represent an alternative to pumped-storage reservoirs for storing large quantities of energy. The peculiarity of CAES is that the gas must be stored under a high pressure (up to 30 MPa) in order to achieve the necessary efficiencies. A lined rock cavern (LRC) in the form of a tunnel or shaft can be used for this pressure range. The internal gas pressure is borne by the rock; while the tightness and stability of the system is guaranteed by a mixed composite lining consisting of a thin steel inner shell and a reinforced concrete outer shell.

Key design issues are (Fig. 1): uplift failure of the overlying rock up to the surface (Section 2); failure and loss of tightness of the steel lining (Section 3); shearing of the plug closing the cavern (Section 4). These issues are investigated in the present paper for 4 m diameter lined CAES tunnels and shafts in various geotechnical conditions.

Fig. 1 Key design issues for a lined CAES rock cavern
Relatively simple computational methods and rock mass models are employed (equivalent continuum models or models with persistent discontinuities). The large and inevitable uncertainties commonly seen in rock mass model definitions can lead to incorrect predictions, even with computational methods that take proper account of the fundamental aspects of failure along pre-existing discontinuities or the propagation of cracks through intact rock.

2. Uplift

Uplift failure in a continuous rock mass model is investigated by numerical stress analyses. Plane strain and axisymmetric models are used for the tunnels and shafts, respectively (Fig. 2). The rock mass is assumed to be a linearly elastic, perfectly plastic, no-tension material obeying the Mohr-Coulomb yield criterion. The analysis consists of the following steps: (i) initialization of the in situ stress with a lithostatic stress distribution; (ii) deactivation of the grid elements inside the tunnel or shaft and application of the average initial pressure; (iii) monotonic increase in the pressure (no cyclical loading) until no equilibrated solution can be obtained.

Perazzelli et al. (2014) and Perazzelli and Anagnostou (2014) performed a similar analysis when investigating uplift in CAES tunnels and shafts in weak rocks. They discussed the behaviour of the model and possible uplift safety criteria with reference to several computational examples. Specifically, they showed that at a certain pressure $p_c$, which is lower than the ultimate pressure $p_{ul}$, the entire zone between the pressurized tunnel and the surface becomes cracked. Even if the bearing capacity of the rock mass is not fully exploited at this state, such situations are best avoided. For this reason we assume here the pressure $p_c$ at which the cracked zone reaches the surface as a limit pressure. In determining this pressure, we observe the development of the cracked zone and the distribution of the tangential stress along the vertical axis under increasing internal pressure (Perazzelli et al., 2014, and Perazzelli and Anagnostou, 2014).

Figure 3 shows the limit pressure $p_c$ as a function of the overburden $H$ for 4 m diameter CAES tunnels and shafts in rock masses with various strength parameters. It should be noted that the rock strength $\sigma_c$ ceases to be decisive for the limit pressure $p_c$ above a certain value of the strength $\sigma_c$, which depends on the overburden. This is because the tensile failure of the rock mass governs the critical pressure $p_c$. The admissible air pressure can be determined by applying a safety factor to the critical pressure $p_c$ (e.g. $F_s = 1.5$). For the given geotechnical conditions, overburdens of 60-80 m are sufficient for avoiding uplift in the case of a 4 m diameter CAES tunnel under an operational pressure of $p = 20$ MPa. For a CAES shaft of the same diameter, the necessary overburden is smaller (40-50 m).

![Fig. 2](image_url)

*Fig. 2* Computational domains and boundary conditions adopted for the numerical stress analysis of (a) a CAES tunnel and (b) a CAES shaft.
In particularly unfavourable rock structures, uplift may occur due to failures along pre-existing discontinuities. In this case, discontinuous models are more appropriate, as they take direct account of the persistence, spacing and shear strength of the discontinuities. We analysed tunnels and shafts in rock masses with horizontal bedding planes and vertical joints (Fig. 4). Such structures are particularly unfavourable but nevertheless typical in sedimentary rocks. We also adopted the limit equilibrium models of Perazzelli and Anagnostou (2014).

The main assumptions of these models are summarized in Figure 4. The horizontal bedding planes are assumed to be continuous and equally spaced (i.e. a constant thickness $t_b$). The vertical joints are assumed to be continuous in the horizontal direction and discontinuous in the vertical direction. Their persistency $p_j$ in the vertical direction is taken greater than the thickness of the beds $t_b$. The spacing $s_j$ and the locations of the joints are treated as stochastic variables. This allows the uncertainties surrounding the actual location of any vertical discontinuities to be considered. A uniform probability distribution is assumed for $s_j$ where the input parameters are the minimum and maximum values of $s_j$. The shear strength of the vertical joints is treated as purely frictional, i.e. their shear strength $\tau = \sigma_c \tan \phi$ (Fig. 4a). The horizontal stress $\sigma_n$ (Fig. 4) is taken equal to the initial lithostatic horizontal stress ($\sigma_n = K_0 \gamma z$). Zero tensile strength is assumed for the bedding planes. The intact rock is assumed to be sufficiently strong for failure to be considered only along the discontinuities.

As a consequence, the failure surface (Fig. 4) is composed of portions of bedding planes and joints. The uplift pressure results from the equilibrium of the rock mass in the vertical direction and it is a function of the given kinematically admissible failure mechanism. The failure mechanism with the smallest volume (marked in grey in Fig. 4) is the one that minimizes uplift pressure. Due to the assumption concerning the spacing $s_j$, the models treat the uplift pressure as a stochastic variable and provide its cumulative probability function as an output (Perazzelli and Anagnostou 2014). In order to obtain a design value for uplift pressure, a limit value must be set for the cumulative probability. We consider here a cumulative probability of 1%.

Figure 5 shows the uplift pressure $p_u$ as a function of the overburden $H$ for 4 m diameter CAES tunnels and shafts in fractured rock masses with different structural parameters. The uplift pressure decreases with decreasing spacing $s_j$ and increasing persistency $p_j$. The safety against uplift increases significantly with the overburden and a shaft is much more favourable than a tunnel as Figure 3 also indicates. In the presence of a particularly unfavourable rock structure, overburdens in excess of 80 m may be required for a 4 m diameter CAES tunnel (at an operational pressure of $p = 20$ MPa), while overburdens in excess of 50 m may be required for a 4 m diameter CAES shaft.
Fig. 4 Discontinuous limit equilibrium models adopted for the uplift analysis of, (a), a CAES tunnel and, (b), a CAES shaft

Fig. 5 Uplift pressure $p_u$ as function of the overburdens $H$ for, (a), a CAES tunnel and, (b), a CAES shaft in a fractured rock mass ($\gamma = 25 \text{kN/m}^3$, $K_0=1$, $\phi=30^\circ$, water table below the CAES utility)

3. Integrity of the steel lining

3.1 Failure modes

We use a simplified plane-strain axisymmetric model for evaluating the interaction between the steel lining and the rock mass under cyclical loading (Fig. 6). The model represents a vertical cross-section of a CAES tunnel or a horizontal cross-section of a shaft. The steel lining is assumed to be in direct contact with the rock mass (no concrete lining) and is considered as an elastic perfectly plastic thin shell (Fig. 6b). The rock mass is considered as an elastic material subjected to an initial uniform and isotropic stress state. Under these assumptions, the lining-rock interaction is the same for tunnels and shafts and is not affected by the depth or the in situ stress $\sigma_0$. This model cannot reproduce the development of irreversible rock deformations and the increase in cavern dimensions observed during cyclical loading (Okuno et al. 2009, Stille et al. 1994), but it can provide an easy explanation of certain key hazard scenarios.

Figure 7a shows a specific example of the relationship between the air pressure $p_{air}$ and the pressure $p_r$ acting on the rock (see Fig. 6b) during a single cycle. Figure 7b shows the tangential stress $\sigma_{tt,s}$ and deformation $\varepsilon_{tt,s}$ of the steel lining during the cycle.
At the start of pressurization (line OA), a small portion of the air pressure $p_{\text{air}}$ is borne by the lining and the remaining part is transferred to the rock. The lining loading $p_{\text{air}} - p_r$ induces tangential tensile stress $\sigma_{tt,s}$ in the steel (Fig. 7b). At a certain air pressure $p_{\text{air}, A}$ (point A), the steel yields ($\sigma_{tt,s} = \sigma_y$). Afterwards (line AB), the air pressure exceeding $p_{\text{air}, A}$ is transferred to the rock. If the maximum air pressure (point B) is too great, then the steel lining fails due to excessive deformation ($\varepsilon_{tt,s} = \varepsilon_{\text{ult}}$) and its tightness is lost.

In order to avoid this type of failure, the air pressure must be lower than the following critical value $p_{\text{lim,1}}$ (Trombetta 2014):

$$p_{\text{lim,1}} = \varepsilon_{\text{ult}} \left( \frac{E}{1 + \nu} \right) + \sigma_y \frac{t}{a},$$

where $a$ is the cavity radius, $t$ the thickness of the steel lining, $E$ and $\nu$ the Young’s modulus and Poisson’s number of the rock, respectively, $\sigma_y$ the yield strength of the steel and $\varepsilon_{\text{ult}}$ the ultimate deformation of the steel.

During depressurization (line BC), both the rock pressure $p_r$ and the lining loading ($p_{\text{air}} - p_r$) decrease. At a certain air pressure, the lining loading and the tangential stress in the steel become zero (point C). Further depressurization (line CD) causes compression of the lining. If the
compression load \( q = p_r - p_{air} \) is too great, then the steel lining fails by buckling. The compression load \( q \) and the subsequent risk of buckling are highest after complete depressurization (\( p_{air} = 0 \), e.g. for carrying out maintenance work in the CAES cavity) and increase with increasing maximum air pressure \( p_{air,B} \). In order to avoid buckling, the maximum pressure \( p_{air,B} \) must be lower than the following critical value \( p_{lim,2} \) (Trombetta 2014):

\[
p_{lim,2} = \left( \frac{\sigma_r}{a} + q_cr \right) \left( 1 + \frac{1}{k} \right), \tag{2}
\]

where the relative lining stiffness

\[
k = \frac{1 + \nu}{1 - \nu} \frac{E_s}{E_a}, \tag{3}
\]

while \( E_s \) and \( v \) denote the Young’s modulus and the Poisson’s number of the steel, respectively, and \( q_cr \) is the buckling pressure of the lining (buckling is analysed in Section 3.2).

The safety against fatigue failure in the steel lining depends mostly on the number of loading cycles, the stress range \( \Delta \sigma_{sc} \) in a single cycle and the type of welding joints. For a given weld type, the maximum allowable stress range \( \Delta \sigma_{sc} \) can be expressed as a function of the number of cycles (see, e.g., BS EN 13445).

According to the adopted simplified model, the stress range in a single cycle \( \Delta \sigma_{ts} \) reads as follows (Trombetta 2014):

\[
\Delta \sigma_{ts} = \left( p_{air,max} - p_{air,min} \right) \left( \frac{k}{k + 1} \right) \frac{a}{t}, \tag{4}
\]

where \( p_{air,max} \) and \( p_{air,min} \) denote the maximum and minimum air pressure, respectively. The air pressure is reduced to zero only for the performance of maintenance work. During normal CAES operation, the minimum air pressure amounts to 2-4 MPa (Johansson 2003). The latter must be considered in the fatigue assessment. According to Eq. (4), the stress range \( \Delta \sigma_{ts} \) increases with increasing maximum air pressure \( p_{air,max} \). In order to avoid fatigue failure in the lining, the maximum air pressure \( p_{air,max} \) must thus be lower than the following critical value \( p_{lim,3} \):

\[
p_{lim,3} = \frac{\sigma_r}{a} \left( \frac{k + 1}{k} \right) \frac{a}{t} + p_{air,min}. \tag{5}
\]

### 3.2 Buckling analysis

We performed nonlinear buckling analyses based on the assumptions below (Fig. 8). The rock was modelled by discrete normal springs with nonlinear behaviour, allowing for rock-lining separation. Fig. 8b shows the assumed relationship between the force and displacement of a spring. The springs react only for outward displacements of the ring, i.e. they can bear only compressive forces. Their stiffness was evaluated as a function of the elastic constants of the rock \((E, \nu)\) using the following relation:

\[
k_{sp} = \left( \frac{E}{1 - \nu} \right) \frac{b}{a}, \tag{6}
\]

where \( d \) is the spacing of the springs (Fig. 8a) and \( b \) is the width of the lining \((d = 0.09 \text{ m and } b = 1 \text{ m})\).

The circular, 4 m diameter lining was modelled by elastic beam elements having a cross-section of
1 m x 0.01 m and, due to the plane strain conditions, a Young's modulus $E_b$ of $E_f/(1-\nu^2)$. A global oval-shaped imperfection was assumed (Fig. 8a), corresponding to the first buckling mode of the linear buckling analysis. A uniform pressure $q$ was assumed to act over the entire lining. Due to symmetry, only a quarter of the entire lining was considered in the numerical model (Fig. 8a).

The analyses were performed using the Abaqus commercial code and the modified Riks method. Figure 9 shows the pressure-deflection curves obtained for various values of the Young’s modulus of the rock. The maxima of the curves correspond to the critical buckling pressures $q_{cr}$. As expected, the buckling pressure $q_{cr}$ decreases with a decreasing Young’s modulus of the rock. The solid line in Figure 10 shows the buckling pressure $q_{cr}$ as a function of the Young’s modulus of the rock. For comparison, the diagram also shows two analytical solutions: the one of Levy (1884), derived for unconfined rings, and the one of Glock (1977), derived for rings embedded in a rigid medium. With decreasing Young’s modulus $E$, the buckling pressure tends asymptotically to a minimum value corresponding to the limit case of an unconfined ring and is very close to the solution of Levy (1884). With increasing $E$, the buckling pressure tends asymptotically to a maximum value corresponding to the limit case of rigid confinement and is close to the solution of Glock (1977). Note that for the typical values of the Young’s modulus $E$ (higher than 1 GPa for rocks), the buckling pressure $q_{cr}$ does not depend significantly on $E$ and is very close to the buckling pressure of a ring embedded in a rigid medium.

![Fig. 8 Computational model of the buckling analysis](image)

![Fig. 9 Pressure q as a function of the displacement u of a control point in the model (a = 2 m, t = 10 mm, $E_s = 198$ MPa, $\nu_s = 0.3$, $\nu = 0.3$)](image)
3.3 Comparison of the different failure modes

Figure 11 shows limit pressures in respect of different failure modes for the steel lining of a 4 m diameter CAES cavity. Line 1 shows failure due to excessive elongation and results from Eq. (1) with $\varepsilon_{ult} = 2\%$. Line 2 shows buckling failure and was computed according to Eq. (2) with the $q_{cr}$ values of Figure 10. Lines 3 and 4 show fatigue failure and result from Eq. (5) with $\rho_{min} = 2\text{ MPa}$ and a maximum allowable stress range $\Delta\sigma_{scr} = 234\text{ MPa}$ or 526 MPa. The limit stress ranges $\Delta\sigma_{scr}$ were determined according to BS EN 13445 assuming 10'000 cycles and two different joint classes (90 for line 3 and 40 for line 4).

The safety against failure increases with increasing stiffness of the rock. Buckling and fatigue are the most relevant failure mechanisms. If the CAES reservoir is not too shallow and the rock is relatively soft, these failure modes are decisive for the maximum operational pressure (see Fig. 11, 3 and 5). CAES at high operating air pressures (> 20 MPa) is unfeasible in relatively soft rocks ($E < 10\text{ GPa}$).

![Fig. 10](image1)

**Fig. 10** Buckling pressure $q_{cr}$ as a function of the Young’s modulus of the rock ($a = 2 \text{ m}, t = 10 \text{ mm}, E_s = 198 \text{ MPa}, \nu_s = 0.3, \nu = 0.3$)

![Fig. 11](image2)

**Fig. 11** Limit pressure in respect of different failure modes for the steel lining as a function of the rock’s Young’s modulus $E$ ($a = 2 \text{ m}, t = 10 \text{ mm}, E_s = 198 \text{ MPa}, \nu_s = 0.3, \sigma_y = 355 \text{ MPa}, \nu = 0.3, \rho_{min} = 2 \text{ MPa}$ for fatigue failure)
4. Stability of the concrete plug

We consider here the particular case of an 80 m deep CAES tunnel of 4 m diameter located in a weak rock mass ($\phi = 30^\circ$, $\sigma_c = 8$ MPa) above the water table and accessed by a service tunnel of equal diameter (Fig. 12a). In this case Perazzelli and Anagnostou (2014) analysed the stability of a rock mass subjected to plug loading by numerical stress analyses based on axisymmetric models (Fig. 12b). They assumed a rock mass comprising a continuous linearly elastic, perfectly plastic, no-tension material obeying the Mohr-Coulomb yield criterion and a concrete plug comprising a linearly elastic material. They used elasto-plastic interface elements for modelling the contact between plug and rock mass. In order to estimate the stability of the rock mass, they evaluated the relationship between the pressure applied to the plug and the displacement of a control point in the model (on the wall of the service tunnel). The limit pressure was taken equal to the pressure at the last equilibrated solution.

Figure 13a shows the limit pressure as a function of the plug dimensions: conicity $t_p$ and length $L_p$ (see the sketch of the figure). For typical operational pressures (up to 20 MPa) the rock mass is stable under the loading of 3 - 6 m long plugs with a conicity of 0.5-1.5 m. A small conicity (e.g. $t_p < 1.5$ m) could be critical if the rock mass close to the tunnel has been significantly disturbed by the excavation and for this reason is not recommended. The plug must be sufficiently long to avoid shear failure of the concrete. For a specific plug geometry, Figure 13b shows the deformed mesh at the last equilibrated solution established. A gap clearly develops between the plug and the rock mass along the interface on the side of the pressurized tunnel. The gap may be critical for the integrity and thus for the tightness of the steel lining (local bending failure) and must be considered in the lining design.

![Fig. 12](image_url) (a) Problem layout. (b) Computational domains and boundary conditions adopted in the numerical stress analyses of the plug ($L_p = 6$ m; $t_p = 1.5$ m) (after Perazzelli and Anagnostou 2014)

![Fig. 13](image_url) (a) Ultimate pressure as a function of the dimensions of the plug. (b) Deformed calculation grid (plug $L_p = 6$ m; $t_p = 1.5$ m) at the ultimate pressure $p = 300$ MPa (after Perazzelli and Anagnostou 2014)
5. Conclusions

In lined tunnels and shafts for compressed air energy storage (CAES), the rock mass and the lining are subjected to very high pressures ($p = 10 - 30 \text{ MPa}$) and cyclical loading. Uplift failure of the overlying rock mass is an important hazard scenario. The results presented show that, with the exception of very unfavourable rock structures, overburdens greater than 50 m allow uplift to be avoided in the case of a 4 m diameter CAES shaft (at an operational pressure of $p = 20 \text{ MPa}$) under the geotechnical conditions considered. CAES tunnels of the same diameter require overburdens greater than 80 m for avoiding uplift.

The safety of the rock mass with respect to uplift failure is a necessary but not a sufficient condition for assessing feasibility. The maximum air pressure should also be kept sufficiently low to preserve the integrity of the lining and, especially, its tightness. Possible failure modes for the steel lining are: tensile failure at the maximum operating pressure, buckling failure at atmospheric pressure (i.e. during maintenance work), fatigue failure under cyclical loading. The safety against these hazard scenario increases with increasing rock stiffness. If the storage utility is not too shallow and if the rock is relatively soft, buckling or fatigue failures of the steel lining are decisive for assessing the maximum admissible air pressure. The results presented show that in soft rocks (e.g. $E < 10 \text{ GPa}$) CAES is not feasible at high operating air pressures ($p_{\text{air, max}} \geq 20 \text{ MPa}$) in 4 m diameter tunnels and shafts sealed with a thin steel shell.

The stability of a rock mass subjected to plug loading was investigated for specific geotechnical conditions and with plug geometries similar to those applied in previous studies (plug length 6 m, conicity 0.5-1.5 m). According to the computational results, rock failure is not a relevant design factor.

Considering that experience with this type of underground openings is very limited and that the inherent model and parameter uncertainties are large, properly monitored field tests in scaled models are essential.

6. References


