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Full stress and displacement fields for steel-lined deep pressure tunnels in transversely anisotropic rock

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ABSTRACT

An analytical solution is derived that provides closed-form formulations for stresses and displacements for a deep pressure tunnel in a transversely anisotropic rock, with a steel liner, and subjected to a uniform internal pressure. For the derivation, it is assumed that the tunnel support includes a thin steel liner, concrete backfill and that there is an annulus of damaged rock around the concrete. It is also assumed that all materials remain elastic and that the concrete and the damaged rock cannot transmit shear or tangential stresses. The solution is verified by providing comparisons between its results and those from the Finite Element program ABAQUS. For thin steel liners, it can be assumed that the contact pressure between the different materials is uniform, and thus the bending moments in the liner are negligible. This is due to the low bending stiffness of the steel liner. The paper is inspired by and expands the work by Pachoud and Schleiss (2015) who conducted an extensive numerical parametric analysis to obtain correction factors that, when used with an analytical solution for isotropic materials, approximate the maximum principal stress in the liner and intact rock.

1. Introduction

The purpose of pressure tunnels is to convey water under pressure with acceptable losses and without causing instabilities. Pressure tunnels are mostly built in hydroelectric power plant projects, but they are also used, albeit with smaller pressures, for water supply and wastewater conveyance.

Pressure tunnels usually have a circular cross section because of its structural advantages due to the large internal pressures, and also for hydraulic reasons. The cross section area and the surface roughness (i.e. unlined tunnel or concrete or steel lining) depend on the head losses accepted over the length of the tunnel, provided that the opening is stable. The location of the ground water table, together with the hydraulic head in the tunnel, determines whether water will flow from the tunnel to the rock or vice versa (Merritt, 1999). If the hydraulic head inside the tunnel is larger than that from the water table, water will flow out from the tunnel, which requires that rock stresses around the opening are larger than the water pressures to prevent hydrofracturing or hydrojacking (Benson, 1989; Seeber, 1985a, b). If the tunnel is supported, the stiffness of the rock determines the contribution of the rock mass to support the internal pressure, as the load transfer from the liner to the rock depends largely on the stiffness of the liner relative to the surrounding rock (e.g. Einstein and Schwartz, 1979; Bobet, 2001, 2003, 2011; Bobet and Nam, 2007). Finally, the permeability of the rock mass is required to estimate the leakage flow and thus the pore pressures behind the support.

The layout of the pressure tunnel can be decided based on empirical methods and the information obtained during field exploration, if available. The empirical methods can provide initial estimates as to where a watertight liner is needed (Dann et al., 1964; Benson, 1989; Hartmaier et al., 1998; Bergh-Christensen, 1982; Broch, 1984a, b; Selmer-Olsen, 1985; Brekke and Ripley, 1989, 1993; Alvarez et al. 1999). Following this, calculations regarding flow rate, stresses in the liner and in the ground should be made to assess whether the leakage is acceptable and that the stress state in the liner and ground is adequate, and to provide an estimate of the factor of safety (see e.g. Schleiss, 1986, 1997; Hendron et al., 1989; Fernandez, 1994; Fernandez and Alvarez, 1994; Bobet and Nam, 2007).

A steel liner is the industry standard for the sections of the tunnel where the minimum principal stress of the rock around the excavation is smaller than the internal tunnel pressure with a suitable factor of safety (Fernandez, 1994). Steel liners are also needed when the internal pressure is large and the surrounding rock has low modulus, so leakage control with reinforced concrete is not

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Nomenclature

Notation

A_s, I_s	cross-section and moment of inertia of the liner
E_c, ν_c	Young's modulus and Poisson's ratio of the backfill concrete
E_{crm}, ν_{crm}	Young's modulus and Poisson's ratio of the damaged isotropic rock
E_{rm}, ν_{rm}	Young's modulus and Poisson's ratio of the intact isotropic rock
E_x^{rm}, E_y^{rm}	Young's modulus of intact rock in x- and y-axis ($E_x = E$ and $E_y = E'$ in Pachoud and Schleiss, 2015)
E_x^{crm}, E_y^{crm}	Young's modulus of damaged rock in x- and y-axis
E_s, ν_s	Young's modulus and Poisson's ratio of the liner
G_{xy}^{rm}	shear modulus of the intact rock
G_{xy}^{crm}	shear modulus of the damaged rock
p_c	radial stress at the steel-concrete contact
p_i	internal water pressure
r, θ	polar coordinates
r_i	radius of the tunnel
r_{crm}	limit of cracked concrete liner
r_{rm}	limit of damaged rock
t_s	liner thickness
T_s, M_s	axial force and moment of the liner
U_x, U_y	displacements in Cartesian coordinates; superscripts s, c, crm or rm denote steel, concrete, damaged rock or intact rock
U_r, U_θ	displacements in polar coordinates; superscripts s, c, crm or rm denote steel, concrete, damaged rock or intact rock
w	gap between steel liner and backfill concrete
x, y	Cartesian coordinates of axes of elastic symmetry
z_k	complex number, $z_k = x + \mu_k y$, $k = 1, 2$
ε_x, ε_y, γ_{xy}	axial and shear strains in x- and y-axis
ε_r, ε_θ, $\gamma_{r\theta}$	radial, tangential and shear stresses in polar coordinates
ζ_k	complex number that depends on z_k through conformal mapping
$\phi(z_k)$, $\phi'(z_k)$	stress function and its derivative
μ_1, μ_2	roots of compatibility equation
ν_{xy}, ν_{xz}, ν_{yz}	Poisson's ratios in x–y axes
σ_x, σ_y, τ_{xy}	normal and shear stresses in Cartesian coordinates; superscripts s, c, crm or rm denote steel, concrete, damaged rock or intact rock
σ_r, σ_θ, $\tau_{r\theta}$	radial, tangential, and shear stresses in polar coordinates; superscripts s, c, crm or rm denote steel, concrete, damaged rock or intact rock

possible. A concrete-encased steel cylinder is the most common type of liner (Eskilsson, 1999). There are two criteria that need to be satisfied for the design of the steel liner (Schleiss, 1988): (1) working stress and deformation of the steel liner; and (2) load-bearing capacity of the rock mass. The first criterion requires a design of the steel liner to adequately support the internal pressures during operation, the external hydraulic pressures during grouting and dewatering, the handling loads during transportation and erection, and that limits local deformations, e.g. crack bridging in the backfill concrete (Schleiss, 1988; Brekke and Ripley, 1993). The second criterion is needed to guarantee sufficient safety against rock failure, and thus ensure the load sharing between liner, backfill concrete and rock assumed for the first criterion.

The design of a steel-lined pressure tunnel due to the internal pressure is generally based on an elastic analysis assuming that the support and rock mass are isotropic and elastic and that the concrete and an annulus of rock surrounding the concrete are damaged such that only radial pressures can be transmitted. Beyond the damaged rock, intact or undamaged rock is assumed (e.g. Schleiss, 1988; Moore, 1989; USACE, 1997; Hachem and Schleiss, 2009; ASCE 2012). Analytical solutions are available that account for these assumptions and are included in the next section for completeness. Pachoud and Schleiss (2015) provided a number of correction factors to the analytical solution to expand it to cases where the intact rock is transversely anisotropic. The factors were obtained after an extensive numerical analysis using a Finite Element Method and a genetic algorithm to minimize errors between the corrected analytical solution and the numerical results. The corrected factors were obtained to estimate the major principal stress in the liner and in the intact rock. This paper builds on the work from Pachoud and Schleiss (2015) and provides a full analytical solution for stresses and displacements for the steel liner, concrete, damaged rock and intact rock. It also expands the range of cases, as the solution accounts for the anisotropy of the damaged rock and does not have the limitation that the existing solution has of assuming that the shear modulus of the rock is equivalent to the empirical relation of Saint–Venant.

Clearly, analytical formulations are limited due to the assumptions that need to be made to reach the solution. In most cases, the design will require a numerical method that does not have the shortcomings of the analytical solutions, as it can consider the construction process, non-linear behavior, etc. Closed-form solutions however are invaluable to obtain a better understanding of the interplay that exists between loads, rock and support, to identify what are the most critical parameters for the problem, and to provide first estimates or even a preliminary design. An added advantage is that they can be used with very little cost to conduct sensitivity analysis and, most importantly, to provide benchmark values to check the results of the more complex numerical models.

2. Steel-lined pressure tunnel in isotropic rock

Fig. 1 shows how the internal pressure is distributed between the steel, concrete and rock mass. As the internal pressure increases, the steel liner may initially take all the stress (this may occur if there is a gap between the steel and the concrete, e.g. due to differential thermal contraction between the steel and the concrete, creep of the rock, cycles of loading and unloading as the tunnel is pressurized and emptied, or the skin grouting of the steel-concrete interface is not done or is not effective). With further increase of internal pressure, as the steel liner expands, part of the loading is transferred to the concrete, to the rock damaged during excavation and to the undamaged/intact rock. It is generally assumed that the concrete and the damaged rock can only transmit radial deformations as the damage prevents transfer of tangential and shear stresses. This results in radial stresses that decrease with $1/r$ within the concrete and damaged rock zones, and $1/r^2$ in the undamaged rock zone. The magnitude of the radial stresses and corresponding radial displacements can be obtained by imposing equilibrium along the radial direction and compatibility of radial displacements at the liner-concrete, concrete-damaged rock, and damaged-undamaged rock boundaries (Moore, 1989; USACE, 1997; Hachem and Schleiss, 2009). Thus:

For the liner:

$$T_s = (p_i + p_c) r_i$$
$$U_r^s = (1 - \nu_s^2) \frac{r_i^2}{E_s t_s} (p_i + p_c) \tag{1}$$