Tunnelling and Underground Space Technology 42 (2014) 227–236

Contents lists available at ScienceDirect

Tunnelling and Underground Space Technology

journal homepage: www.elsevier.com/locate/tust

Pressure tunnels in non-uniform in situ stress conditions

T.D.Y.F. Simanjuntak [a,*], M. Marence [a], A.E. Mynett [a,b], A.J. Schleiss [c]

[a] UNESCO-IHE, Department of Water Science and Engineering, P.O. Box 3015, 2601 DA Delft, The Netherlands
[b] Delft University of Technology, Faculty of CiTG, P.O. Box 5048, 2600 GA Delft, The Netherlands
[c] École Polytechnique Fédérale de Lausanne, LCH-ENAC-EPFL, Station 18, CH-1015 Lausanne, Switzerland

ARTICLE INFO

Article history:
Received 4 March 2013
Accepted 6 March 2014
Available online 1 April 2014

Keywords:
Pressure tunnels
Non-uniform in situ stress
Prestressed lining
Numerical model

ABSTRACT

The bearing capacity of prestressed concrete-lined pressure tunnels is governed by the in situ stress of rock mass, which generally has different magnitudes in the vertical and horizontal direction. Two cases were distinguished, based on whether the vertical stress is greater than the horizontal stress or not.

By means of a finite element model (FEM), the resulting distribution of stresses, strains and deformations due to tunnelling processes was revealed. The bearing capacity of pressure tunnels was determined based on the superposition principle. The results obtained demonstrate the significance of horizontal-to-vertical stress coefficients in the bearing capacity of pressure tunnels prestressed by grouting. Favourite locations where crack openings in the final lining may occur are identified, which is useful for taking measures regarding the tunnel tightness and stability.

1. Introduction

As part of hydropower schemes, the role of pressure tunnels has become significantly more important in maintaining the sustainability of hydropower operation. It is one of the most expensive constructions, especially when the traditional steel linings are used. The need for a more economical design has resulted in a shift from steel to other alternatives, such as prestressed concrete linings. Techniques to enhance the bearing capacity of concrete-lined pressure tunnels embedded in a good quality rock mass have been developed, which can be done either by using an active (Matt et al., 1978) or a passive (Seeber, 1985a,b) prestressing technique.

After the completion of consolidation grouting, a certain prestress level in the final lining can be induced through the injection of a high-pressure cement grout into the circumferential gap between the shotcrete and the final lining (Fig. 1). As well as economic benefits, another reason for the popularity of this passive prestressing technique is that a tight contact between the linings and the surrounding rock mass is achieved, facilitating a continuous load transmission from the final lining to the rock mass and vice versa. In view of a high compressive strength of concrete, the interface grout can be injected at high pressure up to the ultimate limit of concrete strength; however, it should not exceed the smallest principal stress in the rock mass so as to avoid hydrojacking and/or hydrofracturing of the rock mass. The critical point of the design of concrete-lined pressure tunnels hereby has changed from the lining to the rock mass.

As long as rock masses behave as elastic materials and linings are impervious, the Seeber diagram method (Seeber, 1985a) is adequate to estimate the bearing capacity of concrete-lined pressure tunnels prestressed by grouting. Nevertheless, concrete linings are somewhat permeable, rendering the use of a sole load-line diagram can lead to over-prediction of bearing capacity of these tunnels as hydrojacking may occur (Schleiss, 1986). Also, the rock mass in nature is discontinuous, anisotropic, inhomogeneous and non-elastic (Hudson and Harrison, 2001). Depending on their quality, the behaviour of rock masses can be assumed as either elastic-perfectly plastic, elastic-brittle or strain-softening materials (Alejano et al., 2012; Hoek and Brown, 1997). The design of prestressed concrete-lined pressure tunnels features therefore delicate phenomena of both the rock mass and the concrete lining.

Simanjuntak et al. (2012b) have demonstrated the applicability of the finite element model (FEM) to investigate the bearing capacity of prestressed concrete-lined pressure tunnels subject to uniform in situ stress of rock mass. Concurrently, analytical solutions possess great value for conceptual understanding of mechanical and hydraulic behaviour of these tunnels and for model validation. Analytical solutions solving cases of prestressed pressure tunnels subject to non-uniform in situ stress are, however, still lacking and not explicitly revealed; giving that not every problem can be solved using analytical solutions. Here, the use of numerical models are effective in gaining a better understanding

* Corresponding author. Tel.: +31 15 215 1782; fax: +31 15 212 2921.
E-mail addresses: y.simanjuntak@unesco-ihe.org, T.D.Y.F.Simanjuntak@tudelft.nl (T.D.Y.F. Simanjuntak).

http://dx.doi.org/10.1016/j.tust.2014.03.006
0886-7798/

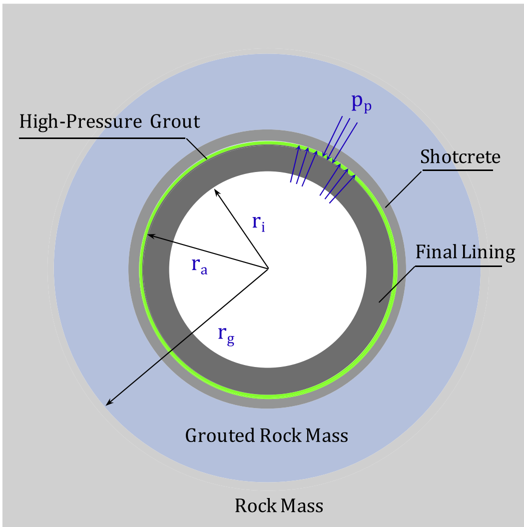

Fig. 1. Cross-section of a prestressed concrete-lined pressure tunnel.

of tunnel behaviour and can promise benefit in many areas from tunnelling to hydropower development.

This paper is concentrated on the predicted bearing capacity of prestressed concrete-lined pressure tunnels subject to non-uniform in situ stress. Regarding the rock mass, the method to predict its bearing capacity is different from the traditional elastic analysis. To be concordant with the authors' previous work, a practical application is dedicated to a deep, straight ahead circular tunnel situated above the groundwater level, embedded in an infinite, homogenous, and isotropic Hoek–Brown rock mass. The rock mass is assumed to behave as an elastic-perfectly plastic non-dilatant material. After the excavation-induced stresses and deformations is obtained, the analysis progresses to the estimation of stresses and deformations transmitted to the support lining, prestress-induced hoop stresses and strains in the final lining, and residual hoop strains in the final lining during the operational water pressure. An approach to estimate the permissible design value of internal water pressure is proposed and further development for the design of prestressed concrete-lined pressure tunnels is outlined.

2. Non-uniform in situ stress of rock mass

In cases where there is no preferred orientation of joints within the rock mass or where the rock mass exhibits non-significant anisotropy in strength and deformability, the Hoek–Brown failure criterion is suitable. The strength of a rock mass according to the Hoek–Brown failure criterion is expressed as (Hoek and Brown, 1980),

$$\sigma_1 = \sigma_3 + \sigma_{ci}\sqrt{m_b\frac{\sigma_3}{\sigma_1} + s} \tag{1}$$

in which σ_{ci} is the uniaxial compressive strength of the intact rock material, σ_1 and σ_3 represent the major and minor principal stress respectively. Parameter constants m_b and s depend on the structure and surface conditions of the joints. They can be evaluated using an empirical index designated as the Geological Strength Index (GSI).

If the variation of vertical loading across the height of excavation is small compared to the magnitude of the stress at the excavation location, the gravitational force can be assumed negligible (Detournay and Fairhurst, 1987). The horizontal stress, σ_h, can be expressed in the product of the corresponding vertical stress, σ_v, and a coefficient of earth pressure at rest, k. The in situ stress is

Table 1
Rock mass properties (Amberg, 1997).

GSI	σ_{ci} (MPa)	m_i	m_b	s	ψ (°)	E_r (GPa)	v_r	σ_o (MPa)
65	75	17	4.87	0.02	0	20.5	0.25	40

non-uniform, if $k \neq 1$. The mean stress, σ_o, can be calculated by the following expressions:

$$\sigma_o = \frac{\sigma_h + \sigma_v}{2} = \frac{k\sigma_v + \sigma_v}{2} = \frac{(k+1)\sigma_v}{2} \tag{2}$$

As a result of excavation works, a plastic zone develops around the tunnel, in which its volumetric behaviour is characterized by a dilation angle, ψ. For cases of plain isotropic rocks, the assumption of non-dilating rock mass, i.e. $\psi = 0$, is appropriate for the prediction of the extent of the plastic zone (Hoek and Brown, 1997; Serrano et al., 2011; Wang, 1996).

3. Tunnel excavation in an elasto-plastic rock

There are two cases distinguished in this paper: a case where the vertical stress is greater than the horizontal stress and another one, where the horizontal stress is greater than the vertical stress. For both cases, a 2 m radius tunnel, R, is excavated in a good quality rock mass having mechanical properties (Table 1) adopted from Amberg (1997). The in situ stress of rock mass is non-uniform with 0° of orientation angle.

Rock mass parameters m_b, s and E_r was obtained using the program RocLab (2002) based on formulae given in Hoek et al. (2002). The implicit FEM implemented in DIANA was used and the structural non-linear analysis was employed so as to simulate the excavation of circular tunnel focussing on the influence of non-uniform in situ stress state and the effects of elasto-plastic rock mass yield on stresses and deformations distribution. A two-dimensional plain strain condition was assumed meaning that the out-of-plane stress coincides with the intermediate principal stress σ_2, and that the problem geometry being analyzed is long and has regular cross-section in the out-of-plane direction (Eberhardt, 2001). The sign convention for compressive stress is negative.

Before the rock mass is excavated, the in situ stress in the rock under consideration is in the equilibrium state and uniformly distributed. After removal of the rock mass within the tunnel, the first deformation occurs. The rock mass around the excavation may deform non-elastically. Connecting the parameters presented in Eqs. (1) and (2), and due to a general property of elasto-plastic continua, according to which the displacements depend linearly on $1/E$ (Anagnostou and Kovari, 1993; Schürch and Anagnostou, 2012), the excavation-induced radial deformations, can be expressed as:

$$\frac{E_r u_r}{\sigma_o R} = f\left(\frac{\sigma_{ci}}{\sigma_o}, k, v, m_b, s, \psi\right) \tag{3}$$

Numerical results of excavation-induced radial deformations are shown in Fig. 2. While Fig. 2a illustrates results of radial deformation distributions after the tunnel excavation for the case where the horizontal-to-vertical stress coefficient $k = 0.80$, Fig. 2b shows the results for $k = 1.25$. For both cases, the representation of radial deformations along the tunnel perimeter is presented in a polar system of coordinates (Fig. 3). Comparing the results between Fig. 3a and b, unlike the radial deformations around a circular excavation subject to uniform in situ stress, radial deformations for cases where the in situ stress are unequal, are non-uniformly distributed.

The shape of underground openings for cases where the in situ stress is non-uniform is considerably different from those for cases in which the in situ stress is uniform. The results in Fig. 3a suggest