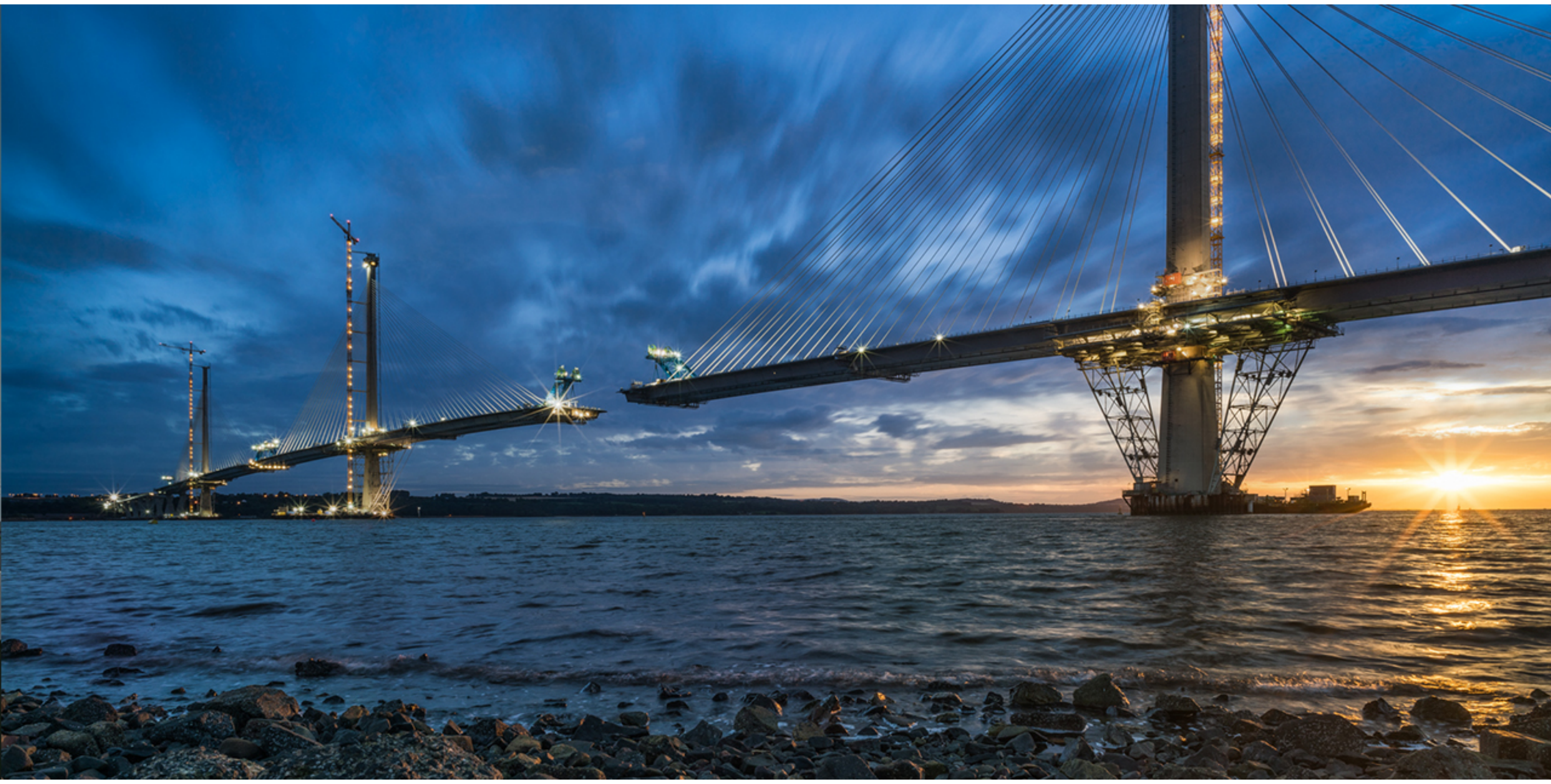

Benchmark Example No. 22

Tunneling - Ground Reaction Line

SOFiSTiK | 2020

VERiFiCATiON
BE22 Tunneling - Ground Reaction Line

VERiFiCATiON Manual, Service Pack 2020-14 Build 134

SOFiSTiK AG

HQ Nuremberg
Flataustraße 14
90411 Nürnberg
Germany

T +49 (0)911 39901-0
F +49(0)911 397904

Office Garching
Parkring 2
85748 Garching bei München
Germany

T +49 (0)89 315878-0
F +49 (0)89 315878-23

info@sofistik.com
www.sofistik.com

Front Cover

Project: Queensferry Crossing | Photo: Bastian Kratzke

Overview	
Element Type(s):	C2D
Analysis Type(s):	STAT, MNL
Procedure(s):	LSTP
Topic(s):	SOIL
Module(s):	TALPA
Input file(s):	groundline_hoek.dat

1 Problem Description

This problem consists of a cylindrical hole in an infinite medium, subjected to a hydrostatic in-situ state, as shown in Fig. 1. The material is assumed to be linearly elastic-perfectly plastic with a failure surface defined by the Mohr-Coulomb criterion and with zero volume change during plastic flow. The calculation of the ground reaction line is performed and compared to the analytical solution according to Hoek [1] [2].

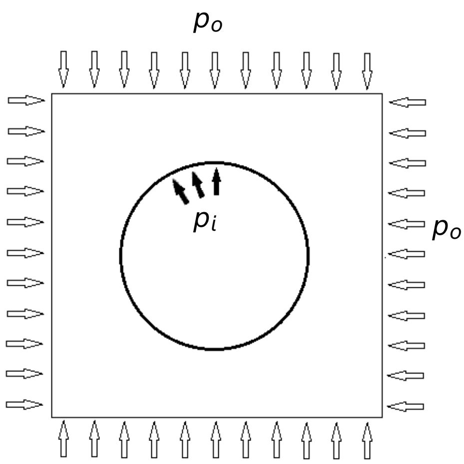

Figure 1: Problem Description

2 Reference Solution

The stability of deep underground excavations depends upon the strength of the rock mass surrounding the excavations and upon the stresses induced in this rock. These induced stresses are a function of the shape of the excavations and the in-situ stresses which existed before the creation of the excavations [1]. When tunnelling in rock, it should be examined how the rock mass, surrounding the tunnel, deforms and how the support system acts to control this deformation. In order to explore this effect, an analytical solution for a circular tunnel will be utilised, which is based on the assumption of a hydrostatic in-situ state. Furthermore, the surrounding rock mass is assumed to follow an elastic-perfectly-plastic material behaviour with zero volume change during plastic flow. Therefore the Mohr-Coulomb failure criterion is adopted, in order to model the progressive plastic failure of the rock mass surrounding the tunnel. The onset of plastic failure, is thus expressed as:

$$\sigma_1 = \sigma_{cm} + k\sigma_3, \tag{1}$$

where σ_1 is the axial stress where failure occurs, σ_3 the confining stress and σ_{cm} the uniaxial compressive strength of the rock mass defined by:

$$\sigma_{cm} = \frac{2c\,cos\phi}{1 - sin\phi}. \tag{2}$$

The parameters c and ϕ correspond to the cohesion and angle of friction of the rock mass, respectively. The tunnel behaviour on the other hand, is evaluated in terms of the internal support pressure. A circular tunnel of radius r_o subjected to hydrostatic stresses p_o and a uniform internal support pressure p_i, as shown in Fig. 2, is assumed.

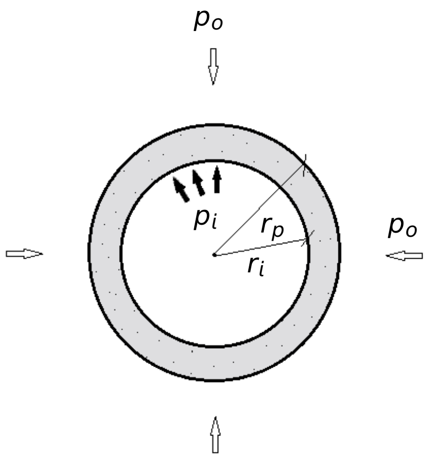

Figure 2: Plastic zone surrounding a circular tunnel

As a measure of failure, the critical support pressure p_{cr} is defined:

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1 + k}, \tag{3}$$

where k is the coefficient of passive earth pressure defined by:

$$k = \frac{1 + sin\phi}{1 - sin\phi}. \tag{4}$$

If the internal support pressure p_i is greater than p_{cr}, the behaviour of the surrounding rock mass remains elastic and the inward elastic displacement of the tunnel wall is:

$$u_{ie} = \frac{r_o\,(1 + \nu)}{E}\,(p_o - p_i), \tag{5}$$

where E is the Young's modulus and ν the Poisson's ratio. If p_i is less than p_{cr}, failure occurs and the total inward radial displacement of the walls of the tunnel becomes:

$$u_{ip} = \frac{r_o(1+\nu)}{E}\left[2(1+\nu)(p_o - p_{cr})\left(\frac{r_p}{r_o}\right)^2 - (1-2\nu)(p_o - p_i)\right], \tag{6}$$

and the plastic zone around the tunnel forms with a radius r_p defined by:

$$r_p = r_o\left[\frac{2(p_o(k-1)+\sigma_{cm})}{(1+k)((k-1)p_i+\sigma_{cm})}\right]^{\frac{1}{(k-1)}} \tag{7}$$

3 Model and Results

The properties of the model are defined in Table 1. The Mohr-Coulomb plasticity model is used for the modelling of the rock behaviour. The load is defined as a unit supporting pressure, uniform along the whole line of the circular hole, following the real curved geometry. The ground reaction line is calculated, which depicts the inward oriented deformation along the circumference of the opening that is to be expected in dependence of the acting support pressure.

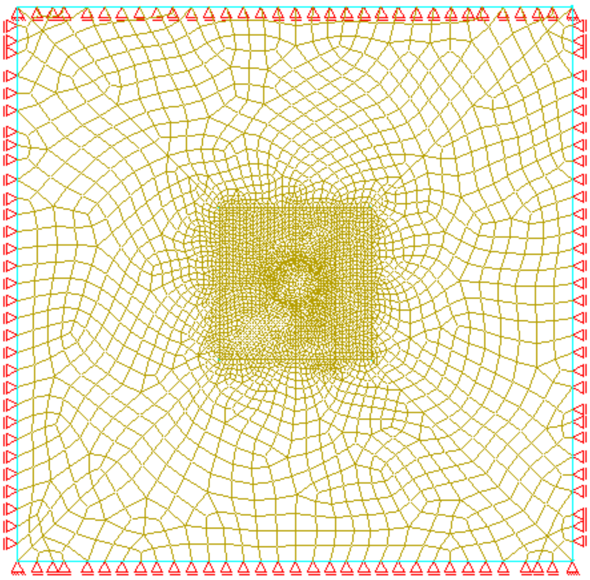

Figure 3: Finite Element Model

Table 1: Model Properties

Material Properties	Geometric Properties	Pressure Properties
$E = 5000000\, kN/m^2$	$r_o = 3.3\, m$	$P_o = 29700\, kN/m^2$
$\nu = 0.2$		$P_{i_{max}} = 7000\, kN/m^2$
$\gamma = 27\, kN/m^3$		$P_{cr} = 8133.744\, kN/m^2$
$\gamma_{buoyancy} = 17\, kN/m^3$		

Table 1: (continued)

Material Properties	Geometric Properties	Pressure Properties
$\phi = 39°$, $\psi = 0°$		
$c = 3700\, kN/m^2$		
$k = 4.395$		

The uniaxial compressive stress of the rock mass σ_{cm} is calculated at $15514.423\ kN/m^2$ and the critical pressure p_{c_r} is $8133.744\ kN/m^2$. The ground reaction line is presented in Fig. 4, as the curve of the inward radial displacement over the acting support pressure. It can be observed that the calculated values are in agreement with the analytical solution according to Hoek.

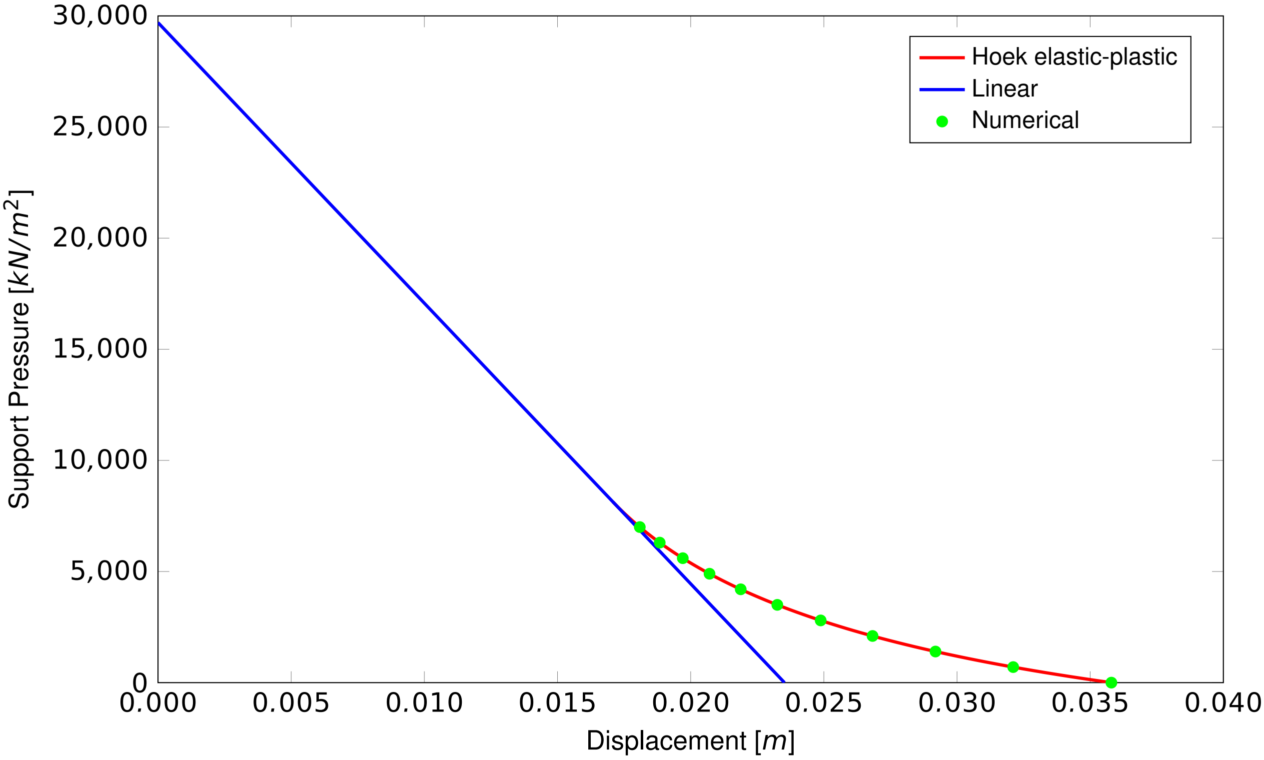

Figure 4: Ground Reaction Line

4 Conclusion

This example examines the tunnel deformation behaviour with respect to the acting support pressure. It has been shown that the behaviour of the tunnel in rock is captured accurately.

5 Literature

[1] E. Hoek. *Practical Rock Engineering*. 2006.
[2] E. Hoek, P.K. Kaiser, and W.F. Bawden. *Support of Underground Excavations in Hard Rock*. 1993.