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Equivalent Mohr–Coulomb and generalized Hoek–Brown strength parameters for supported axisymmetric tunnels in plastic or brittle rock

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Abstract

The evaluation of equivalent Mohr–Coulomb (M–C) strength parameters to the prototype Hoek–Brown (H–B) ones for tunnels has been tackled in different ways for many years. The extension of the H–B criterion to the generalized one has made the challenge even greater. Most of the latest methods did not account for the effect of the support pressure and none gave formulae for equivalent parameters of supported or brittle rock. Here, an almost exact explicit solution for the evaluation of the critical pressure, of a tunnel in a rock mass satisfying the generalized H–B criterion, is initially investigated. Then, formulae are derived for the evaluation of equivalent parameters, of either elastoplastic or elastic–brittle plastic rock. They are based either on a best fitting procedure of the two envelopes or on the equation of selected responses of the models. Supported tunnels in equivalent M–C rock masses are then validated against those excavated in the prototype H–B rock masses.

Keywords: Tunnel; Equivalent strength parameters; Axisymmetry; Brittle; Plastic; Supported rock mass

1. Background

Various empirical criteria have come up over the past decades in an attempt to simulate triaxial behavior of the rock mass. They are based on large scale testing, experience and back analysis. Of these, the Hoek–Brown (H–B) non-linear criterion, which has been the most widely used, has seen several changes over the years. It is an extension of the intact rock criterion by mitigating the strength of the latter according to the quality of the rock mass. This quality is quantified with the use of the geological strength index GSI. Its application has to be linked with good geological judgment and adequate classification of the rock mass.

An overall design methodology involves evaluation of the H–B intact rock constants and classification of the rock mass quality according to GSI, in order to evaluate the rock mass strength parameters to use them as input for detailed numerical analysis of difficult and complicated construction projects. However, most numerical codes do not accept such a non-linear criterion and may allow only for linear criteria, such as the Mohr–Coulomb (M–C). This necessitated the development of guidelines for the evaluation of M–C parameters from the prototype H–B ones, which when used in numerical codes for the simulation of constructions in rock, would evaluate similar responses. In this way, neither the advantages of simplicity of the linear M–C criterion nor of improved rock representation of the H–B criterion, are lost. Such M–C parameters and corresponding rock masses are defined as equivalent to the prototype H–B ones, respectively.

The generalized exponential form of the H–B criterion [1] for the rock mass (Fig. 1), which is its last version, is given by

$$\sigma_{1N} = \sigma_{3N} + (m_b\sigma_{3N} + s)^a, \qquad (1)$$

σ_{1N} and σ_{3N} are the major and minor principal stresses normalized with respect to the unconfined compressive strength σ_{ci} of the intact rock, and m_b, s and a are rock

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Nomenclature

BFa	best fit procedure in a variable artificial stress range
BFe	best fit procedure in the existing stress range
EMR	equating model responses procedure
a	Hoek–Brown constant for the rock mass
c	cohesion
C_0	uniaxial compressive strength for a rock mass described by the Mohr–Coulomb criterion
D	disturbance factor
GSI	geological strength index
H–B	Hoek–Brown
k	gradient of the Mohr–Coulomb criterion defined in terms of principal stresses
M–C	Mohr–Coulomb
$m_{\rm i}$	Hoek–Brown constant for intact rock
$m_{\rm b}$	Hoek–Brown constant for the rock mass
p_0	natural field stress
$p_{\rm e}$	critical pressure (radial stress at the outer boundary of the plastic zone) for a rock mass described by the generalized (exponential) H–B strength criterion
$p_{\rm e0}$	critical pressure (radial stress at the outer boundary of the plastic zone) for a rock mass described by the original (parabolic) H–B strength criterion
$p_{\rm i}$	support pressure
R	radius of underground opening
$r_{\rm e}$	radius of the outer boundary of the plastic zone in the rock mass around a tunnel
s	Hoek–Brown constant for the rock mass
u	radial displacement
W	Lambert's W function

Greek letters

ϕ	angle of internal friction
$\sigma_{\rm ci}$	uniaxial compressive strength of the intact rock
$\sigma_{\rm cm}$	global "rock mass strength" [1]
$\sigma_{\rm t}$	strength of the rock mass in "biaxial tension" [1]
ψ	dilation angle

Indexes

N	the stress or pressure is normalized with $\sigma_{\rm ci}$
R	residual parameters

mass strength parameters given by

$$m_{\rm b} = m_{\rm i}\,\exp\left(\frac{\rm GSI - 100}{28 - 14D}\right), \tag{2}$$

$$s = \exp\left(\frac{\rm GSI - 100}{9 - 3D}\right), \tag{3}$$

$$a = \tfrac{1}{2} + \tfrac{1}{6}\left({\rm e}^{-{\rm GSI}/15} - {\rm e}^{-20/3}\right), \tag{4}$$

$m_{\rm i}$ is an intact rock constant and D is a rock mass disturbance factor.

The linear M–C strength criterion in the τ–σ space is described by the c and ϕ parameters representing the rock mass cohesion and friction angle, respectively. In the σ_1–σ_3 space, the criterion (Fig. 1) depends on the parameters C_0 and k, where C_0 is the unconfined compressive strength of the rock mass and k is the gradient. It is defined by

$$\sigma_1 = k\sigma_3 + C_0, \tag{5}$$

$$k = \frac{1 + \sin\phi}{1 - \sin\phi}, \tag{6}$$

$$C_0 = \frac{2c\cos\phi}{1 - \sin\phi}. \tag{7}$$

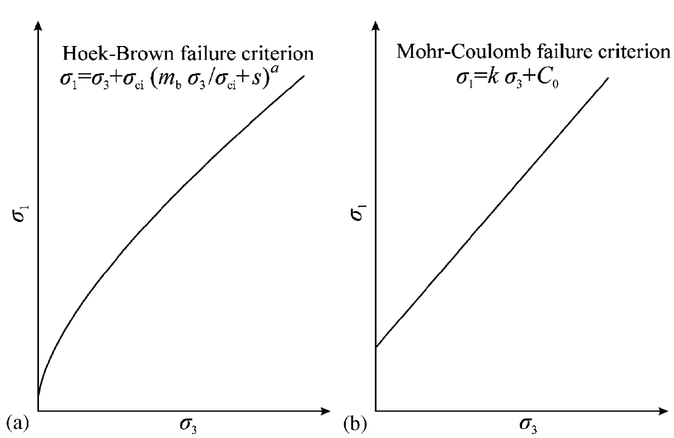

Fig. 1. Non-linear Hoek–Brown (a) and linear Mohr–Coulomb (b) failure criteria.

Due to the highly non-linear nature of the H–B criterion for low values of the minor principal stress, determination of equivalent M–C strength parameters for any physical problem involves a procedure of linearization within a desired stress range. Early considerations of this problem, were based upon tangents [2] to the H–B envelope in the τ–σ space. In a subsequent stage, derivation of the equivalent M–C parameters was based on average values determined by curve fitting over a predetermined constant range of the minor principal stress from zero to the quarter of the intact rock strength [3]. This latter assumption allowed for the development of two charts which provided unique values of c and ϕ for given GSI, $m_{\rm i}$ and $\sigma_{\rm ci}$ parameters. It was however recognized that for shallow tunnels this range should extend to the overburden pressure value. Later, Sofianos and Halakatevakis [4] realizing that the constant range used to determine the