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# Tunnel Support Using the Convergence Confinement Method: RBD and Partial Factor Calibration

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**Abstract:** This paper presents results of investigations on the applicability of RBD and partial factor calibration to tunnel support design using the convergence confinement method. We show how partial factors could be developed for specific cases and levels of reliability using well-developed RBD methods, and that such factors produce designs of the prescribed reliability level. However, we also expose significant challenges in practically developing such factors. In particular, we demonstrate that limited understanding of the variability of rock mass parameters make defining structure classes arbitrary, which leads to code format partial factors that are meaningless. Additionally, we use the results of these investigations to raise other questions regarding future development of geotechnical design codes.

Keywords: RBD; partial factors; convergence-confinement; rock engineering.

## 1 Introduction

The rock mechanics community is currently investigating application of reliability-based design (RBD), primarily due to the introduction of geotechnical design codes that are based on it (e.g., CSA 2014; CEN 2004). It is now known to be theoretically applicable to tunnel (e.g., Lü et al. 2011) and slope design (e.g., Jimenez-Rodriguez et al. 2006). However, customary rock engineering design techniques do not use the partial factor approaches commonplace in geotechnical design codes. Some preliminary investigation into the development of partial factors for the support of underground rock wedges has been performed (El Matarawi and Harrison 2017), and this demonstrated the feasibility of calibrating and using factors in some specific cases. Here, the more complex problem of tunnel support using the convergence-confinement method (CCM) is investigated.

## 2 Reliability-Based Design and Partial Factor Calibration

### 2.1 Principles of reliability-based design

Application of RBD requires the following:

1. Identification of the random variables (RVs), defined by random vector  $\mathbf{X} = [X_1, X_2, \dots, X_n]$ , where  $X_i$  is a random variable that governs the behaviour of the structure;
2. Functional definition (a limit state function, LSF) of the failure state of structure of the form  $g(\mathbf{X}) = 0$ , where  $g(\mathbf{X}) \geq 0$  defines the region of random variable space where performance of the structure is satisfactory, and  $g(\mathbf{X}) < 0$  defines the region where performance is unsatisfactory (i.e. failure occurs);
3. Setting a target probability of failure  $P_{ft}$  for the design.

In the context of the simplest LSF of  $g = R - S$ , where  $R$  is the resistance and  $S$  is the load, these requirements are shown in Figure 1a. Analytical calculation of  $\Pr(R < S)$  by integration of the joint PDF of  $R$  and  $S$  is generally impossible for structures, and so it is common to use the approximate first-order reliability method (FORM) of Hasofer and Lind (1974), which transforms the system into standardized and uncorrelated normal space (SUNS), and finds the point on  $g(R, S) = 0$  with the greatest probability of occurrence. This is termed the design point  $\mathbf{u}_d$ , with the distance between this and the mean of the PDF being  $\beta$ , the reliability index. The probability of failure is  $P_f = \Phi(-\beta)$ . A closed-form solution for  $g(R, S)$  is required to calculate  $\beta$ , and in its absence other approaches are required. Due to its intuitiveness the response surface method (RSM) is popular, and is outlined here; this is further discussed in Section 3.

### 2.2 Calibrated partial factors

For each  $X_i$ , the design value  $x_{d,i}$  is related to the characteristic value (say, the mean)  $x_{k,i}$  of the marginal distribution by a partial factor  $\gamma_{X_i} = x_{d,i}/x_{k,i}$ . Unlike global factors of safety, which are set empirically, partial factors may be calibrated from RBD such that deterministic analysis using values of  $x_d$  produces designs that

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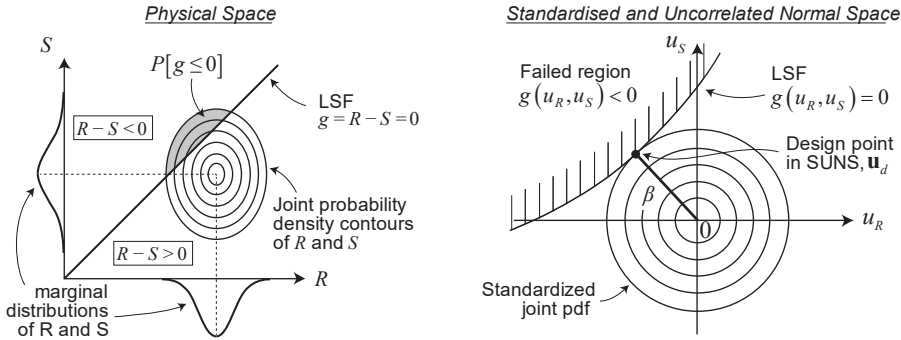
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possess a specific probability of failure. The magnitude of the partial factors reflects both the uncertainty in the random variables as well as their sensitivity on the probability of failure.



(a) Components for RBD of LSF  $g=R-S=0$  in physical space (b) Definition of reliability index  $\beta$

Figure 1. RBD of simple LSF  $g=R-S=0$

It is conventional in structural engineering to define “resistance” (i.e. favourable) partial factors as  $\gamma_{x_i} = x_{k,i} / x_{d,i}$  and “load” (i.e. unfavourable) factors as  $\gamma_{x_i} = x_{d,i} / x_{k,i}$ , in order that factors are greater than unity. However, in geotechnical and rock engineering the distinction between “favourable” and “unfavourable” variables is often not clear and so, as discussed by Low and Phoon (2015) and El Matarawi (2016), we use the definition  $\gamma_{x_i} = x_{d,i} / x_{k,i}$  throughout.

3 Application of RBD to Tunnel Lining Design Using CCM

CCM considers the interaction of the stress-displacement characteristics of both the ground in which the tunnel is being excavated (the so-called ground response curve, GRC) and the tunnel lining (the support reaction curve, SRC) (Figure 2). Here, we use the CCM formulation of Carranza-Torres and Fairhurst (2000) together with a formulation for pre-excavation tunnel convergence (Vlachopoulos and Diederichs 2009). This formulation assumes a homogeneous, isotropic, non-dilating, time-independent, continuum rock mass whose strength is defined by the Hoek-Brown criterion, together with full-face sequential excavation under hydrostatic far-field in-situ stresses. For the tunnel lining, we examine unreinforced concrete with a linearly-elastic brittle response.

3.1 Input parameters

Many of the input parameters in CCM could be regarded as random variables, but here we assume variability in only the rock mass elastic modulus  $E_r$ , the hydrostatic in-situ stress  $p_0$ , and the rock mass strength captured through the residual standard error  $\epsilon_{HB}$  from regression of the Hoek-Brown peak strength criterion (Bozorgzadeh and Harrison 2015). In order to approximate a bounded normal distribution, the 4-parameter beta distribution with mean  $\mu$ , standard deviation  $\zeta$ , lower bound  $a$  and upper bound  $b$  (e.g., Lü et al. 2011) is used to characterize these variables (Table 1). The bounds are chosen to either represent physical constraints or  $\pm 10\zeta$ . The values of the random variables are given in Table 2, and the deterministic variables are shown in Figure 2.

3.2 Limit state functions and fitting procedure

Figure 2 shows that either radial displacement or support pressure could be used as limiting conditions. Consequently two limit state functions were considered in this case study:  $g_1$ , concerning radial displacement and with a limiting value of 2%, and  $g_2$ , representing the support capacity:

$$g_1 = 2\% - (u_{eq} / R_0) \text{ and } g_2 = p_{c,max} - p_{eq} \tag{1}$$

Both of these conditions represent the coordinates of the intersection of the GRC and SRC, and as these cannot be written explicitly, the response surface method was used to evaluate them. Second order response surface functions with two factor interactions,

$$\tilde{g}_1 \text{ or } \tilde{g}_2 = b_0 + b_1(E_r) + b_2(\epsilon_{HB}) + b_3(p_0) + b_4(E_r, \epsilon_{HB}) + b_5(E_r, p_0) + b_6(\epsilon_{HB}, p_0) + b_7(E_r^2) + b_8(\epsilon_{HB}^2) + b_9(p_0^2), \tag{2}$$

were used to approximate them. This was established using a forward-selection approach to ensure a sufficiently simple yet comprehensive function that satisfies the requirements of linear regression. The structure of the

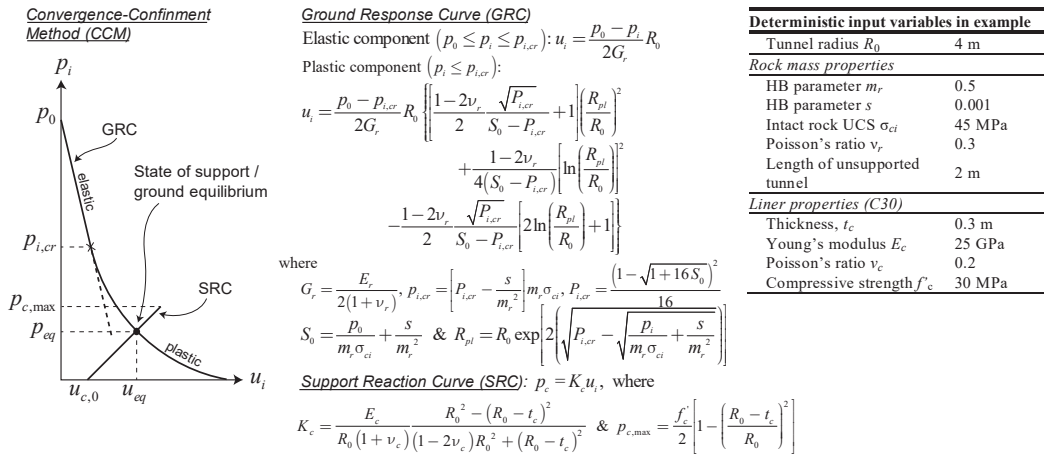
response surface is consistent with those recommended in literature. A central composite design structure comprising three levels of 2<sup>n</sup> factorial and star designs, in addition to a central point, was adopted. This resulted in 43 points for every response surface fitting, thereby ensuring reliable function estimates even when some points within the design structure resulted in conditions of no convergence between the GRC and SRC.

**Table 1.** Beta distribution parameters.

Lower bound, $a$	Upper bound, $c$	Shape parameter, $\lambda_1$	Shape parameter, $\lambda_2$
For $E_r$ and $p_0$ : $\max(0, \mu - 10\zeta)$	For $E_r$ and $p_0$ : $\mu - 10\zeta$	$\lambda_2(a - \mu) / (\mu - c)$	$[(\mu - c)(\mu^2 + \zeta^2 - \mu a - \mu c + ac)] / \zeta^2(a - c)$
For $\varepsilon_{HB}$ : $\max(-\sigma_{ci}\sqrt{s}, -10\zeta_{HB})$	For $\varepsilon_{HB}$ : $+10\zeta_{HB}$		

**Table 2.** Random variables.

	$\mu$	$\zeta$	Coefficient of variation $\Omega$	4-parameter beta distribution			
				$a$	$c$	$\lambda_1$	$\lambda_2$
Rock mass elastic modulus $E_r$	0.5 GPa	0.1 GPa	0.2	0	1.5	16.3	32.7
HB error $\varepsilon_{HB}$	0 MPa	0.142 MPa	-	-1.423	1.423	49.7	49.7
Hydrostatic in-situ stress $p_0$	13 MPa	2.6 MPa	0.2	0	39	16.3	32.7



**Figure 2.** Convergence-confinement method formulation and deterministic input parameters used in case study.

**3.3 Analysis and results of RBD**

Table 3 summarises the results for a lining thickness of 0.30 m. Convergence to the design points within a tolerance of 1% was achieved within 2 iterations for both LSFs, validating the efficacy of the response surface function and the fitting scheme. For the displacement LSF  $g_1$ , with  $t_c = 0.30$  m and an unsupported tunnel length of 2 m, the design point has  $\beta = -2.864$  and  $P_f = 99.79\%$ . The target values were  $\beta_f = 2.5$  and  $P_f = 0.62\%$ , and so this design is unsatisfactory. On the other hand, LSF  $g_2$  for support capacity is satisfied with a resulting  $P_f$  of 0.34%. These values of  $P_f$  were verified using Monte Carlo simulation (MCS);  $P_f$  computed using MCS for  $g_1$  and  $g_2$  are 99.69% and 0.20% respectively. The difference between the two sets of results is due to FORM being an approximation; if necessary, second-order reliability method (SORM) may give a more accurate estimate.

**Table 3.** Results of RBD for case study considering the support capacity LSF.

LSF	Design point in SUNS $u_d = (E_{r,d}, \varepsilon_{HB,d}, p_{0,d})$	$\beta$ ( $P_f$ )	$\alpha = (E_r, \varepsilon_{HB}, p_0)$
$g_1$ ; radial displacement	(1.179, 0.033, -2.610)	<b>-2.864</b> (99.79%)	(0.412, 0.011, -0.911)
$g_2$ ; support strength	(-0.248, 0.499, 2.653)	<b>2.711</b> (0.34%)	(0.092, -0.184, -0.979)

Table 3 also shows the computed direction cosines  $\alpha$ , which indicate the sensitivity of the RVs on  $\beta$ . Both LSFs are most sensitive to the in-situ stress  $p_0$ , with the negative sign indicating that increasing in-situ stress creates more unfavourable conditions. This sensitivity demonstrates the paramount importance of accurately characterizing the in-situ stress for tunnel design. Interestingly, the sensitivity of rock mass elastic modulus and rock mass strength is seen to be different for  $g_1$  and  $g_2$ :  $g_1$  is more sensitive to  $E_r$  than  $\varepsilon_{HB}$ , whereas the reverse is true for  $g_2$ . Also, the sign associated with  $\varepsilon_{HB}$  is seen to be different for  $g_1$  and  $g_2$ . These

differences confirm the difficulty in geotechnical problems of characterising variables as either favourable or unfavourable. Finally, we note that the reported sensitivities of the RVs on  $\beta$  are only specific to the case investigated here, and may vary depending on the design conditions.

**4 Partial Factor Calibration and Design**

For the purpose of demonstrating the how partial factors may be calibrated and applied in deterministic design, we examine LSF  $g_2$  for support strength. Recalling the partial factor definition  $\gamma_{x_i} = x_{d,i}/x_{k,i}$  and considering the mean values  $\mu_i$  as the characteristic values  $x_{k,i}$ , the computed partial factors for LSF  $g_2$  are summarized in Figure 3. For  $E_r$  and  $p_0$ , this partial factor definition can be applied. However, a more appropriate definition for the factoring of  $\epsilon_{HB}$  is to define a design curve using percentiles (Bozorgzadeh and Harrison, 2015):

$$\epsilon_{HB,d} \text{ Percentile} = \Phi\left(\epsilon_{HB,d}/\zeta_{HB}\right) \times 100\% \tag{3}$$

where  $\epsilon_{HB,d}$  and  $\zeta_{HB}$  are the design values computed from RBD and the standard deviation for the HB error term respectively, and  $\Phi$  is the standard normal cumulative density function. A computed 69th design percentile means that 69% of the observed variability in peak strength at a given  $\sigma_3$  as defined by the random variable  $\epsilon_{HB}$  is below the design strength curve; this is schematically depicted in Figure 3. Methods of calculating the HB design curve are presented in Bozorgzadeh and Harrison (2015) and El Matarawi (2016).

Recall that the objective of calibrated partial factors is to allow deterministic calculation to produce designs of prescribed probabilities of failure without the need for full reliability analysis. The designer applies the partial factors to the characteristic values of the random variables, and performs a deterministic analysis to achieve a design which satisfies the LSF—in this case,  $g_2 = p_{c,max} - p_{eq} \geq 0$ . Typically a design code would require the engineer to design for one variable at a time, but in the case of the tunnel considered here, two design variables require consideration: the length of unsupported tunnel and the lining thickness. Hence, for a given set of ground conditions, a code should provide partial factors for different lengths of unsupported tunnel, and the designer would use the set of partial factors most appropriate for their construction sequence.

Assume that for logistical reasons the designer requires a support delay of 2 m (as assumed in the study here). They would select the following partial factors and design percentile values:  $(\gamma_{E_r}, \gamma_{p_0}) = (0.95, 1.59)$  and  $\epsilon_{HB,d}$  percentile = 69th, and then proceed with calculating RV design values:

$E_{r,d} = \gamma_{E_r} E_{r,k}$	$p_{0,d} = \gamma_{p_0} p_{0,k}$	$\epsilon_{HB,d} = \zeta_{HB} \left[ \Phi^{-1}(\epsilon_{HB,d} \text{ percentile}) \right]$
$\rightarrow E_{r,d} = (0.95)(0.5 \text{ GPa})$	$\rightarrow p_{0,d} = (1.59)(13 \text{ MPa})$	$\rightarrow \epsilon_{HB,d} = (0.142 \text{ MPa}) \left[ \Phi^{-1}(0.69) \right]$
$\rightarrow \boxed{E_{r,d} = 0.46 \text{ GPa}}$	$\rightarrow \boxed{p_{0,d} = 20.67 \text{ MPa}}$	$\rightarrow \boxed{\epsilon_{HB,d} = 0.070 \text{ MPa}}$

A comparison with Figure 3 shows that, barring rounding errors, these are equivalent to the design values calculated from RBD. With these values, the designer would perform a deterministic design using the convergence-confinement method, or a comparable method, iteratively changing the design variable (i.e the lining thickness) until  $g_2 = p_{c,max} - p_{eq} \geq 0 \rightarrow p_{c,max} \geq p_{eq}$  is satisfied. Considering lining thicknesses of 0.2 m, 0.3 m and 0.35 m, the results of the convergence-confinement method are shown in Figure 4. It is clear that a thickness of 0.2 m does not achieve a state of equilibrium. On the other hand, a thickness of 0.3 m most economically satisfies  $g_2 = 0$  compared to a thickness of 0.35 m. This clearly demonstrates that partial factors may be used to obtain a design identical to that given by RBD. By performing this analysis, the designer would ensure that their design satisfies a prescribed probability of failure—in this case, a minimum of 0.62%.

**5 Partial Factor Code Calibration**

The objective of code calibration is to produce sets of partial factors for “structure classes”—structures which are mechanically and statistically similar—in order that members within the structure class are designed to specific objectives and levels of reliability. To perform such optimization, representative cases within the structure classes are selected, RBD is performed to obtain designs that satisfy the target reliability, and then partial factors calculated for each of case. These partial factors will vary somewhat from one another, and so finally one set of partial factors is chosen to represent them all; this may be either the mean partial factor, the most conservative of all the cases, or an optimized value that minimizes the difference between the reliability achieved using the partial factors and the target reliability. Gayton et al. (2004) discuss such procedures.

The concept of structure classes distinguished on the basis of similar behaviour and statistical values has not been previously considered in rock engineering. It requires extensive data collection to characterize the variability of the mechanical properties of rock masses, the uncertainty in the measurement technique, and the

uncertainty of the models used for design (Kulhawy and Phoon 2002). Something that resembles the concept of structure classes that finds extensive use in rock engineering design is that of rock mass classes obtained by application of rock mass classification systems (e.g. the Q system; Barton et al. 1974). Although these schemes give guidance with respect to similarly behaving rock mass groups for specific design applications (Bieniawski 1973), they lack the statistical rigour required for code calibration.

	$E_r$	$\varepsilon_{HB}$	$p_0$
Design value $x_{d,i}$	0.47 GPa	0.0712 MPa	20.61 MPa
Characteristic value $x_{k,i}$	0.5 GPa	0 MPa	13 MPa
<b>Partial factors</b>			
$\gamma_{X_i} = x_{d,i} / x_{k,i}$	<b>0.95</b>	-	<b>1.59</b>
<b>Design percentile</b>	-	<b>69%</b>	-

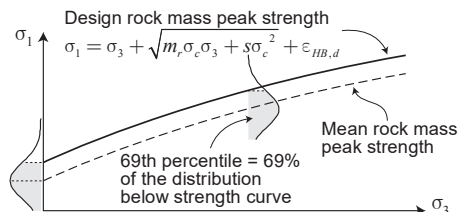


Figure 3. Computed partial factors for LSF  $g_2$  for case study, and definition of rock mass peak strength design percentile.

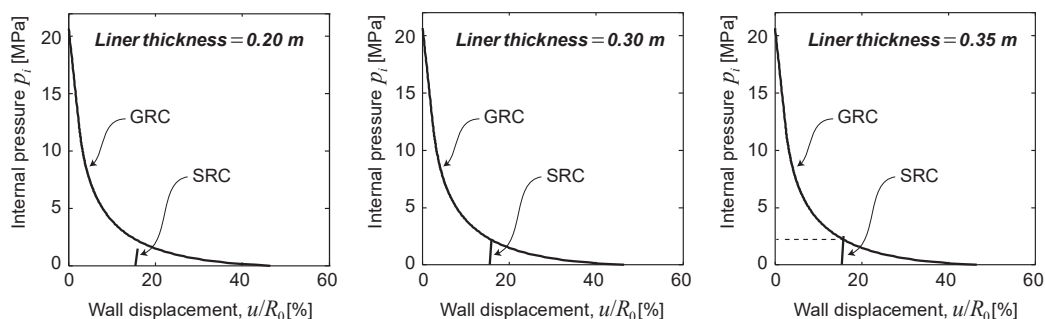


Figure 4. Design of tunnel lining with partial factors.

Given the lack of suitable structure classes, here an initial class was defined based on plausible ranges of values for each of the model parameters, and divided into reasonable segments. RBD was then performed for all possible combinations of values with the objective of determining a lining thickness that satisfied  $\beta_T = 2.5$  for LSF  $g_2$ ; the values used are summarized in Table 4, and this resulted in 255,150 cases. The calculated partial factors are plotted in Figure 5 and for clarity of presentation are shown in two 2D plots instead of a single 3D plot. Each pair of points represents one unique combination of input parameters. Note that the cases plotted in Figure 5 only represent 20.2% of the 255,150 cases assessed; these were the only cases for which RBD was successful in calculating a design. Most of the remaining 79.8% of cases failed due to insufficient points for the response surface fitting to return a solution.

Table 4. Values used in the code calibration procedure.

Rock mass properties	Values considered	Hydrostatic in-situ stress	Values considered
Elastic modulus mean (GPa)	[0.5, 2, 5, 10, 15]	Mean (MPa)	13
HB parameter $s$	[0.001, 0.01, 0.1]	Coefficient of variation	[0.1, 0.2, 0.3]
Intact UCS (MPa)	[5, 15, 25, 35, 45]		
Poisson's ratio	0.3		
Elastic modulus coeff. of variation	[0.1, 0.2, 0.3]	<b>Support delay</b>	Values considered
HB parameter $m_r$	[0.5, 1, 4, 7, 10, 13, 16]	Distance behind tunnel face (m)	[1, 2, 3]
Rock mass UCS coeff. of variation	[0.1, 0.2, 0.3]		

Figure 5 clearly shows that the computed partial factors do not fall within a sufficiently small range to allow one set of factors to be used to represent the many conditions. The wide distribution of values indicates that many smaller subsets would need to be used, but this would result in many structure classes and hence many sets of values of partial factors. The end result would be a very cumbersome design code.

## 6 Discussion and Conclusion

As with previous investigations concerning underground rock wedge support (El Matarawi and Harrison 2017), this work has revealed that calibrated partial factors can be obtained for and used in deterministic design of tunnel linings with the convergence-confinement method. However, the preliminary investigations performed

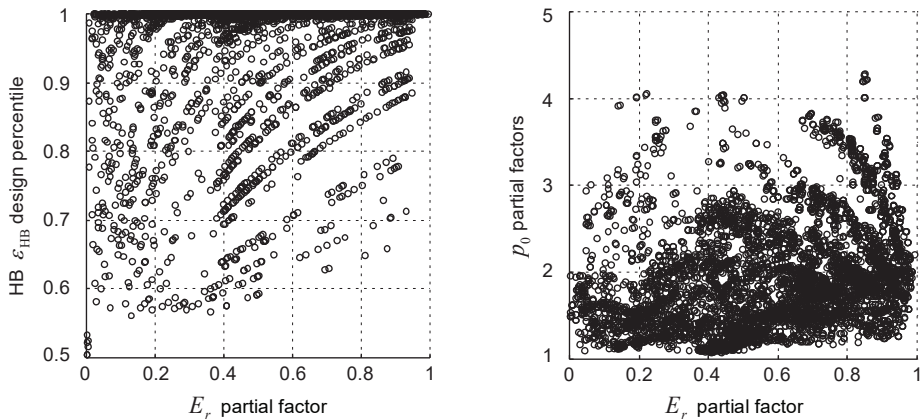


Figure 5. Partial factors computed for support capacity LSF  $g_2$ .

aimed at partial factor code calibration have revealed that poorly-defined structure classes hinders further development. Many questions are raised regarding the potential use of calibrated partial factors in design codes for tunnel lining design, in particular the following:

1. Are partial factor approaches – which assume sufficient understanding of the variability of the conditions—appropriate, considering that many structures are designed with only limited understanding of the variability of rock mass conditions? Should regionally-specific partial factors be developed by regulatory agencies given the local understanding of the geology? Should codes embrace advanced Bayesian techniques (e.g. Bozorgzadeh et al. 2018) to augment limited site-specific data?
2. Should case-by-case full reliability analysis be prescribed for tunnel design, given the existing limitations of partial factor development and the high-risk nature of these structures? Correspondingly, should codes require that designers demonstrate competency in full RBD, or should codes allow simplified RBD methods (e.g. the spreadsheet method of Low and Tang 1997)?

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