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Reliability analysis of tunnel final lining

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ABSTRACT: Tunnel final linings are usually designed according to the methods and safety levels required by the Eurocodes. These codes are mainly applicable in conventional structures, where variability of the permanent loads is mainly due to the uncertainty of unit weights. On the contrary, the loads on the final lining of tunnels result from the interaction of the surrounding rock mass with the temporary support and the final lining. Therefore, they are subjected to much larger uncertainty as the geotechnical properties of the rock mass and the calculation model of the structural interaction both involve appreciable uncertainty. This paper investigates the variation of final lining loads using Monte Carlo simulation for the variability of the rock mass geotechnical parameters. The analyses show that the coefficient of variation of the loads is 20% - 50%, appreciably larger than the usually assumed typical value of 10% corresponding to the self weight and other permanent loads. As a result, reinforced concrete tunnel lining sections designed according to the partial factors of the Eurocodes have appreciably larger probability of failure than conventional reinforced concrete structures. The paper finally calculates the required modification of the partial factors for tunnel linings to achieve different reliability levels (e.g. the same reliability level with the conventional structures).

Keywords: Tunnel, final lining loads, Monte Carlo simulation, partial factors, reliability analysis

1 INTRODUCTION

Plain or reinforced concrete tunnel final linings are designed to undertake loads such as pressure from the surrounding geomaterials and groundwater, live loads, accidental loads (e.g. explosion, fire), temperature and seismic loads. Among them, the most important is the ground pressure applied directly (due to ground creep) or indirectly (due to long-term deactivation of the temporary support). The magnitude of this load depends on the interaction of the surrounding ground with the temporary support and the final lining and is influenced by the construction sequence and especially the time interval between the construction of temporary support and final lining in case of geomaterials with time dependent behaviour.

The final lining of tunnels is usually designed according to the methodologies and the partial factors proposed by the Eurocodes. These codes are mainly applicable in conventional structures, where variability of the permanent loads is mainly due to the uncertainty of unit weights. On the contrary, the loads on the final lining of tunnels result from the interaction of the surrounding rock mass with the temporary support and the final lining. Therefore, they are subjected to much larger uncertainty as the geotechnical properties of the rock mass and the calculation model of the structural interaction both involve appreciable uncertainty. The large uncertainty of the geotechnical parameters and the lack of a widely approved methodology for the design of the tunnel final lining have led to conservative designs with “hidden” safety factors, such as the empirical methods for the estimation of tunnel lining loads and the very conservative assumption of complete de-activation of all temporary support measures in the long-term. Consequently, failure incidents of tunnel final lining are rare, but the reason is an over-conservatism in the design rather than good understanding and modelling of the mechanisms involved.

This paper investigates the ground loads on the final lining of tunnels through probabilistic methods. In the first part the coefficient of variation of tunnel loads from the surrounding rock mass is estimated using empirical and analytical methods through Monte Carlo simulation. The second part calculates the values of the partial factors of permanent loads to achieve different reliability levels. Analogous probabilistic approaches in tunnel excavation and loading are presented in the papers of Papaioannou et al. (2009), Mollon et al. (2009), Courage and Vervuurt (2009) and Fortsakis et al. (2010).

2 ESTIMATION OF FINAL LINING LOAD VARIATION

2.1 *Description of probabilistic analyses*

The factors controlling the uncertainty of tunnel final lining loads are the geometrical parameters (tunnel section, depth of overburden etc), the properties of the surrounding ground and construction materials (strength and deformability, including their long-term behaviour) and the empirical, analytical and numerical models used (e.g. constitutive models, failure criteria of the rock mass, etc). Since it is impossible to take into account all these factors, the present paper concentrates on the most important among them, which is the variability of the geotechnical properties of the rock mass surrounding the tunnel.

The rock mass properties are described using an elastic-perfectly plastic model following the Hoek-Brown failure criterion (Hoek et al., 2002). The model parameters are determined via empirical correlations with rock mass index properties such as the Geological Strength Index (GSI), the uniaxial compressive strength of the intact rock σ_{ci} and the rock-type constant m_i. The probabilistic characteristics of these parameters were originally quantified in the present paper taking into account suggestions from the literature (Hoek, 1998; Baecher, 1983; Park et al. 2005).

The coefficient of variation of GSI was determined from the density of the isolines in the GSI chart (Marinos and Hoek, 2000; Marinos et al., 2005). The range was assumed to be s=±5, for GSI values lower than 30, s=±7 for GSI between 30 and 40 and s=±10 for GSI values larger than 40. In the case where GSI distribution was assumed uniform the scatter defined the upper and lower limits and in the case where the distribution was assumed normal it defined the 90% confidence interval, leading to the calculation of the standard deviation. The coefficient of variation of σ_{ci} (assuming truncated normal distribution) was chosen in accordance with the values proposed for the cohesion (c), the undrained shear strength (S_u) and the unconfined compressive strength of soil formations in the literature (Harr, 1987; JCSS, 2001a; Kuhlway, 1992; Fredlund and Dahlman; 1971; Schultze 1971). The coefficient of variation of m_i (assuming truncated normal distribution) was calculated based on the values proposed by Marinos and Hoek (2001) assuming that the scatter corresponds to 90% confidence interval. The values of $V_{\sigma ci}$ and V_{mi} were decreased by 20% to take into account in an indirect way the spatial variation (El Ramly et al., 2002); whereas this decrease was not applied to GSI since it is a macroscopic parameter which corresponds to a large volume of rock mass. The rock mass deformation modulus was calculated according to the relationship proposed by Hoek et al. (2002) and the rock mass deconfinement due to tunnel face advance was determined based on the curves proposed by Chern et al. (1998).The values for all deterministic and probabilistic parameters used in the parametric analyses are presented in Table 1.

Final lining loads were estimated through widely used empirical and analytical methods: Terzaghi empirical method (Terzaghi, 1946), Unal (1983), Protodyakonov (1948), Terzaghi analytical method (Terzaghi, 1943) and convergence - confinement method (Duncan Fama, 1993). Although these methods are based on different assumptions and thus give a wide range of results (Fortsakis, 2009), they can provide a representative range for the coefficient of variation (V_p) of the final lining loads.

Table 1. Range of parameters for the final lining load probabilistic analyses.

Parameters	Range / Values
Overburden height	H = 20-300m
Tunnel section radius	R = 8, 10m
Geostatic stress ratio	K = 0.5-1.5
Rock mass unit weight	$\gamma = 0.025 MN/m^3$
GSI	Distribution: Normal, Uniform, m_{GSI} = 10-70 & σ_{GSI}: it depends on m_{GSI}
Intact rock uniaxial compressive strength	Distribution: Truncated normal, $m_{\sigma ci}$ = 4-30MPa & $V_{\sigma ci}$ = 25%
Material constant	Distribution: Truncated normal, m_{mi} = 6, 10 & V_{mi} = 10%, 16% (depends on m_{mi})
Disturbance factor	D = 0
Rock mass Poisson ratio	$\nu_m = 0.30$
Shotcrete thickness	d_{shot} = 0.20m
Shotcrete elastic modulus	E_{shot} = 20GPa
Shotcrete Poisson ratio	$\nu_{shot} = 0.20$

2.2 *Probabilistic analyses results*

The results of the stochastic analyses for all the empirical and analytical methods are presented in Fig. 1 as a function of GSI mean value or the mean geotechnical conditions quantified via the factor σ_c/p_o (σ_c=2ctan(45+φ/2), Mohr-Coulomb uniaxial compressive strength and p_o=γH, vertical geostatic stress). Terzaghi empirical method and Unal method lead to values of V_p from 5% to 25%. Protodyakonov method, due to the load mechanism adopted, results to a narrower range than the other two analytical methods (V_p=30%-45%) for most of the analyses. In the case of Terzaghi analytical method the distribution of V_p as a function of $m_{\sigma c}/p_o$ is “radial” since each “radius” corresponds to a different value of overburden height (V_p=10%-80%). Finally according to the results of the convergence - confinement method the coefficient of variation V_p lies between 10% and 80%.

Figure 2 illustrates the distribution of V_p as a function of the mean normalized load for the Terzaghi analytical method and the convergence – confinement method. It becomes evident that the large values of V_p correspond to low values of mean load, which is a result of the proportional large decrease of the denominator of V_p.

The very high values of variation (>60%) are not taken into account, since they correspond to favourable geotechnical conditions where the final lining loads are small and the values of V_p are much more sensitive to the computational procedure. For example in the Terzaghi analytical method, in case of high overburden, the silo mechanism leads to very low final lining load, resulting to unrealistically large values of V_p. Moreover the “weighting factor” of the analytical methods is larger than the one of the empirical methods, which are much simpler and take into account only one parameter. As far as the type of distribution of the load, it varies in respect with the method and the geotechnical conditions from symmetrical to highly non-symmetrical. Finally in the second part of the paper the distribution of the tunnel loads was assumed to be normal and the range for the load coefficient of variation is V_p=20%-50%.

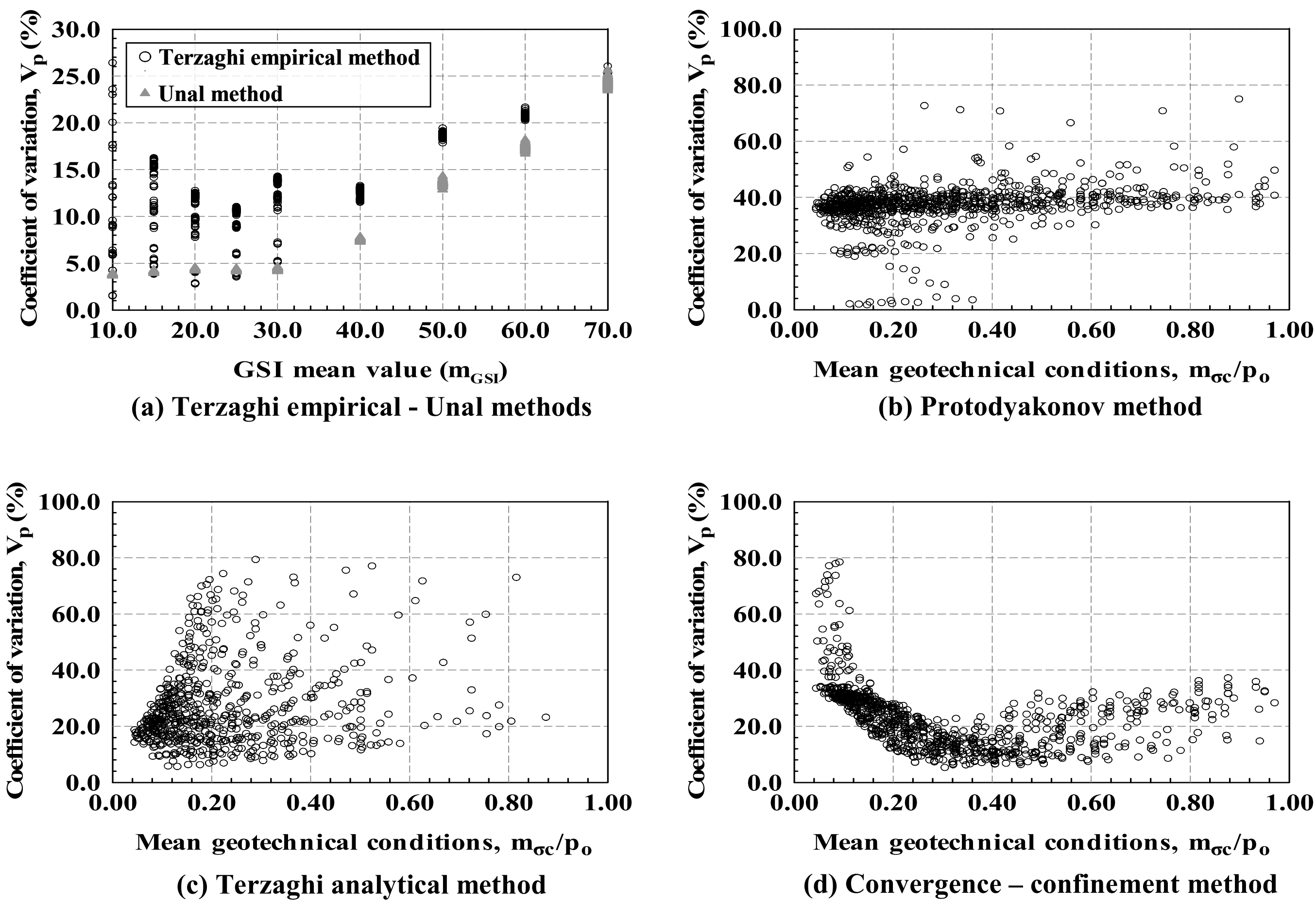


Figure 1. Calculation of coefficient of variation of tunnel final lining loads (V_p) through empirical and analytical methods and Monte Carlo simulation.

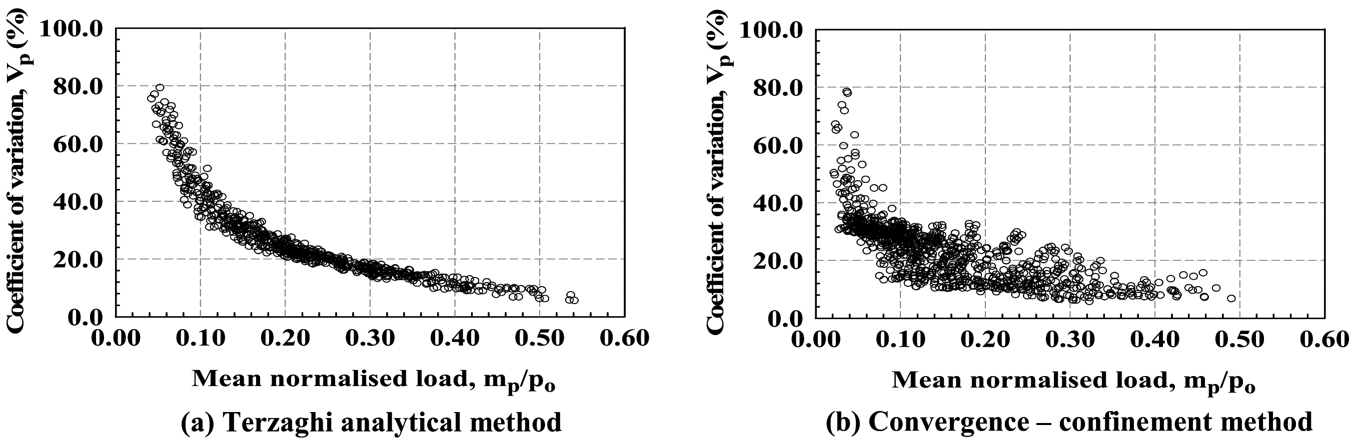


Figure 2. Coefficient of variation of tunnel final lining loads as a function of the mean normalized load.

3 RELIABILITY ANALYSIS OF FINAL LINING

3.1 *Description of probabilistic analyses*

According to Haar (1987) and JCSS (2001b) the coefficient of variation of the unit weight (main permanent load in conventional structures) can be assumed equal to 10%, significantly lower than the range calculated for the tunnel loads in the previous paragraph. The scope of the probabilistic analyses, in the second part, is to estimate the probability of failure (reliability index) of typical final lining sections constructed with reinforced concrete, subjected to loads with different coefficients of variation in terms of axial force and bending moment and consequently calculate the required partial factors, in order to achieve specific reliability levels.

The parameters used in the probabilistic analyses are presented in Table 2. The strength of concrete and steel were assumed to follow normal distribution and the coefficients of variations were set according to the suggestions of Araujo (2001) and Thomos and Trezos (2006). The variation of the axial force and moment has been assumed equal to the variation of the final lining load, since they are considered to be a result of this load only (the influence of live, accidental and other permanent loads is disregarded). Additionally this admission is reasonable as the design is based on elastic analyses. Bending moment has been expressed in terms of eccentricity $m_M = m_N \times e$.

Table 2. Parameters for the reinforced concrete probabilistic analyses.

Parameters	Range / Values
R/C section width	b_{RC} = 1.00m
R/C section height	h_{RC} = 0.30 – 1.00m
Concrete compressive strength	Distribution: Normal f_{ck} = 20, 25, 30MPa, V_c = 10%
Steel yield strength	Distribution: Normal f_{yk} = 400, 500MPa, V_s = 5%
Final lining load coefficient of variation (According to the results presented in the previous paragraph)	V_p = 10% – 50%
Mean value of axial force	m_N =0.10 – 8.00MN
Axial force eccentricity	e/h_{RC} = 0, 0.20, 0.40

Figure 3 illustrates the interaction diagram of a specific reinforced concrete section designed according to the partial factors proposed by Eurocodes. The range between the uniaxial strength in tension and compression has been divided into 1000 increments. For each one of the axial force values, the distribution of M_R is determined, through Monte-Carlo simulation (considering f_c and f_y as random variables). The probability of failure is equal to $p_f=p(M_R<M_{sd})$. It is evident that the breadth of the 90% confidence interval increases as the axial force increases since the participation of concrete, which has larger coefficient of variation than steel, is larger. Yet, in Figure 3-b it is shown that the reliability index β ($\beta=Erf^{-1}(1-p_f)$) is larger in the area of compression than the area of tension, due to the relatively large partial factor of concrete. The highest values of reliability index are calculated around the area of maximum moment (ν_d=0.40-0.60).

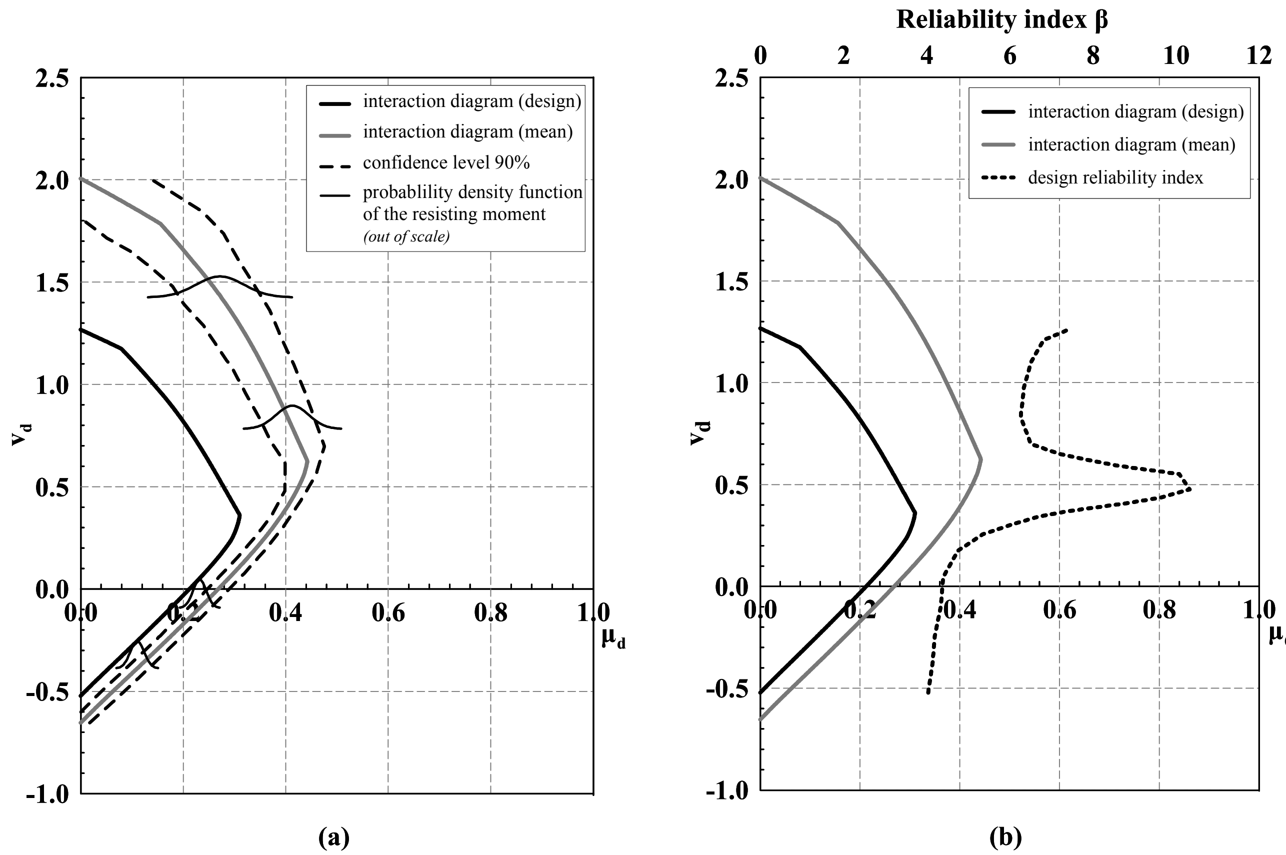


Figure 3. (a) Design and probabilistic interaction diagrams of a specific reinforced concrete section (b) Distribution of reliability index as a function of normalised axial force v_d (b_{RC}=1.00m, h_{RC}=0.50m, A_S=40cm^2 per side, f_{ck}=25MPa, f_{yk}=500MPa, V_p=0.10).

3.2 *Probabilistic analyses with Eurocodes partial factor for permanent loads*

Initially the reliability level of the typical reinforced concrete sections for the different values of V_p is calculated. The value V_p=10% corresponds to the conventional structures and the values 20% to 50% to tunnel loads. It is noted that the value of V_p affects not only the probabilistic calculations but also the deterministic since it differentiates the characteristic values.

- Initial input parameters: b_{RC}, h_{RC}, m_N, e, V_N=V_M=V_p, f_{ck}, V_c, f_{yk}, V_s.
- Calculation of the mean value of concrete and steel strength (α corresponds to 95% percentile according to Eurocodes).

$$m_c = \frac{f_{ck}}{(1+aV_c)}, \; m_s = \frac{f_{yk}}{(1+aV_s)} \qquad (1)$$

- Calculation of the mean value of bending moment: m_M=m_N×e.
- Calculation of the characteristic values of axial force and bending moment (α corresponds to 95% percentile).

$$N_k = m_N(1+aV_N), \; M_k = m_M(1+aV_M) \qquad (2)$$

- Calculation of the design values of all the parameters according to the partial factors proposed in Eurocodes.

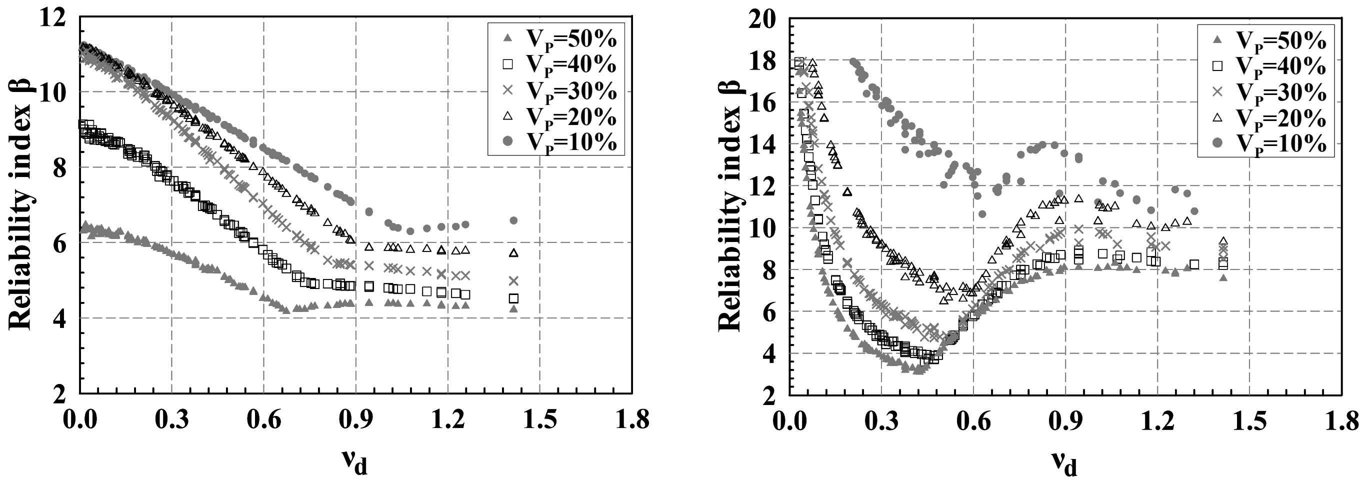
$$N_d = \gamma_g N_k = 1.35N_k, \; M_d = \gamma_g M_k = 1.35M_k, \; f_{cd} = f_{ck}/\gamma_c = f_{ck}/1.50, \; f_{yd} = f_{yk}/\gamma_s = f_{yk}/1.15 \qquad (3)$$

- Calculation of the required reinforcement (A_s), which is considered symmetrically constructed. The minimum reinforcement is considered 0.008×b_{RC}×h_{RC}.
- Calculation of the probability of failure / reliability index of the reinforced concrete section considering the axial force, bending moment and strength parameters as stochastic variables following normal distribution through Monte Carlo simulation with 60000 iterations. The number of iterations leads to satisfactory convergence of the results.

The axial force and the bending moment are presented in terms of the normalised factors ν, μ. It is noted that the horizontal axis corresponds to the ν_d values calculated with $V_{p=}10\%$. Although values of ν_d larger than 1.00 are considered very high, they are plotted to illustrate the trend of the curves as axial force increases.

$$\nu = \frac{N}{bhf_{cd}}, \ \mu = \frac{M}{bh^2 f_{cd}} \qquad (3)$$

According to the results of the parametric analyses (Figure 4) the reliability index decreases until a local minimum which corresponds to the maximum value of ν_d for which the minimum reinforcement is sufficient. This point depends on the values of V_p and e and it is not the same for all the curves because the horizontal axis corresponds to the ν_d of V_p=10%. Moreover in the case of e=0.20h_{RC} the distribution of β after the local minimum is qualitatively similar to the distribution of β in Figure 3-b. It must be noted that the reliability indexes are generally high especially compared to geotechnical problems.



(a) e=0.00×h_{RC} (b) e=0.20×h_{RC}

Figure 4. Distribution of reliability index as a function of design normalised axial force for two different values of eccentricity. (f_{ck}=25MPa, f_{yk}=500MPa). The horizontal axis (design value of the normalised axial force ν_d) corresponds to V_p=10% and γ_g=1.35.

3.3 *Calculation of permanent load partial factors for different reliability levels*

Based on the same methodology described for the probabilistic analysis in the previous paragraph, an iterative analysis is performed, in order to calculate the requisite partial factors of permanent loads for the following cases. During this procedure the dimension of each section studied remain constant and the additional strength required from the increased partial factor, that leads to the requisite reliability level is achieved through additional reinforcement. The results are presented in Figure 5 as a function of the normalised axial force ν_d for V_p=10%.

- Reliability index β = 4.26 – 5.61 (probability of failure (10^{-8} to 10^{-5}).
- Reliability index β equal to the reliability level of the same section which corresponds to loads with V_p=10%.

The values of the partial factors increase as the coefficient of variation and the reliability level increase. In the case that the requirement is the reliability level to be equal to the one corresponding to V_p=0.10, the resulting partial factors are very high, since the probability of failure was very low as it was discussed in the previous paragraph. Moreover the partial factors proposed by Eurocodes seem to be sufficient for the lower range of variation examined.

The distribution of the partial factors is similar to the reliability index distribution in Figure 4, whereas the local maximum of the partial factor diagram coincides with the local maximum of the reliability index one. This is presented in detail in Figure 6 where both variables have been plotted in the same diagram.

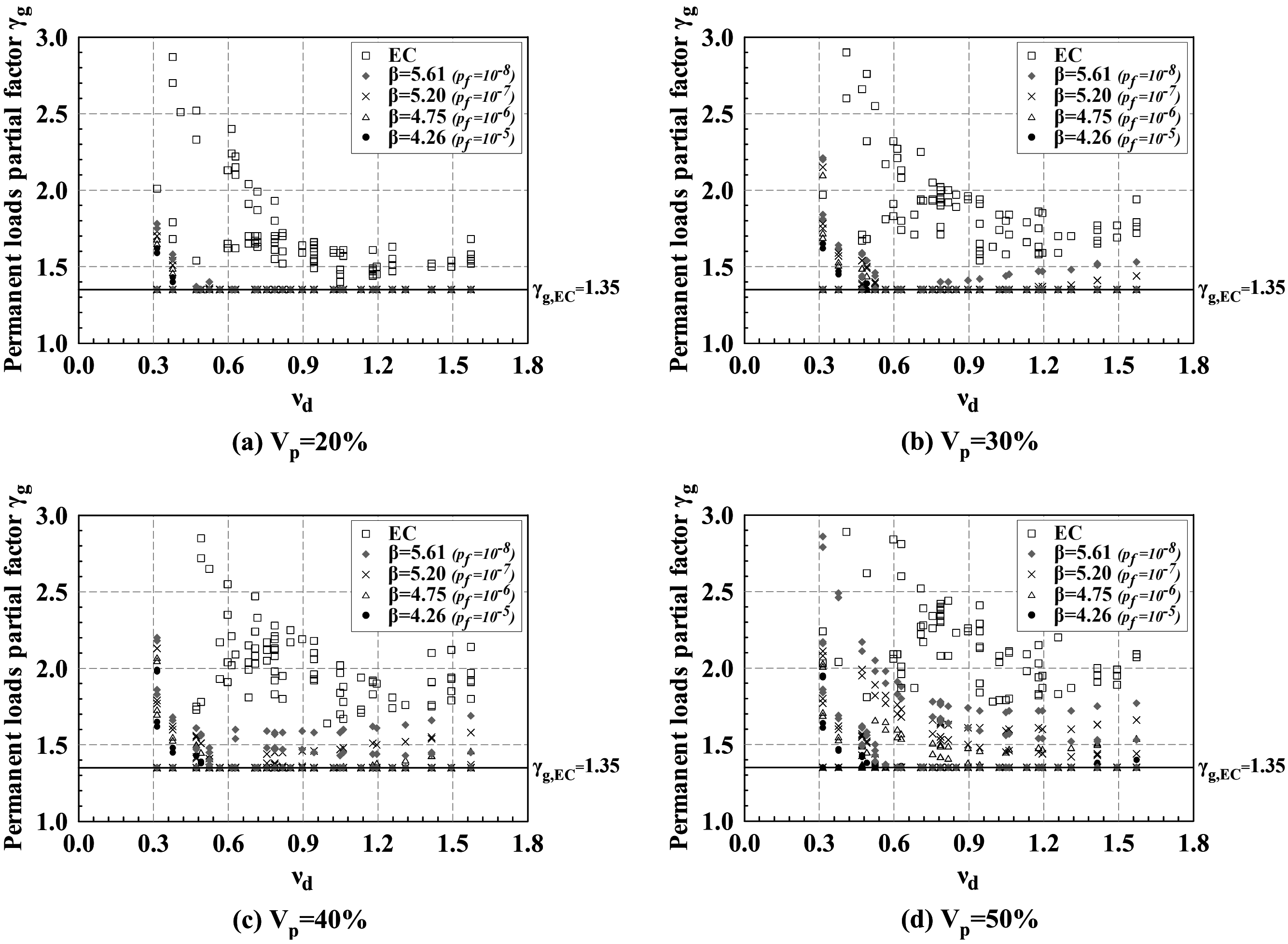


Figure 5. Requisite value of permanent load partial factor as a function of V_p and the reliability level. The symbol EC stands for the case where the demand is the reliability index to be equal with the corresponding of the case V_p=10% and γ_g=1.35. The horizontal axis (design value of the normalised axial force ν_d) corresponds to V_p=10% and γ_g=1.35.

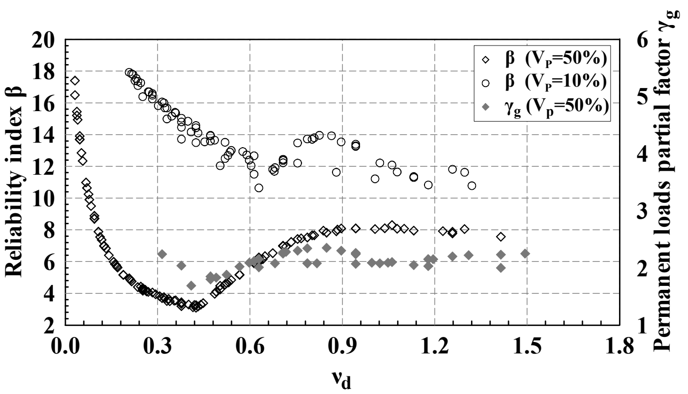


Figure 6. Distribution of reliability index β and calculated permanent loads partial factor γ_g as a function of ν_d (f_{ck}=25MPa, f_{yk}=500MPa). The horizontal axis (design value of the normalised axial force ν_d) corresponds to V_p=10% and γ_g=1.35.

4 CONCLUSIONS

The design of the final lining of tunnels has much larger uncertainties compared to conventional structures, because the applied loads result from the interaction between the surrounding ground, the temporary support and the final lining and, furthermore, the ground parameters include significant uncertainty. The paper performs probabilistic analyses using empirical and analytical methods for the estimation of the tunnel loads and concludes that the corresponding coefficient of variation V_p is in the range 20% - 50%. Actually, this range may be even larger, since not all the factors affecting the uncertainties of tunnel loading can be incorporated in probabilistic analyses. These values are much larger than the corresponding values for conventional structures.

Consequently, when the design of the final lining of tunnels is performed using the partial factors for conventional structural elements (e.g. those required by the Eurocodes), the design results in lower reliability level. Based on this conclusion, the required values of the partial factors for permanent load were determined for certain reliability levels. In the case of large coefficients of variation and high reliability level, the required partial factors increase significantly compared to the values proposed in the Eurocodes (1.35) and can range even between 2.0 - 2.5. The required increase of the partial factors leads to a corresponding increase in the steel reinforcement, which can be large in some cases. Because of that, and since ground variability cannot be reduced, there is a need to rationalise the design of final lining, establish more accurate design methods for the calculation of ground loads on tunnel linings and thus maintain a high reliability level combined with low cost.

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