Non-linear Structural Analysis and Safety Estimation of Natural Draft Cooling Towers Using ECOV Method

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Keywords: Non-linear Analysis, Cooling Tower, Safety Concept, ECOV Method.

ABSTRACT: Non-linear analysis of natural draft cooling towers has been subject of research during the last decades. While sophisticated analysis tools such as elements and constitutive laws have been developed and implemented in finite element codes in the past, special attention must be paid in the development and application of design code consistent safety formats in future. By the introduction of the first Eurocode generation almost 20 years ago, the semi probabilistic safety concept has been established in structural design. This means that in common linear elastic design, factorized load combinations are applied and section design forces are compared with factorized section design resistance. However, in non-linear analysis, a given system is checked whether it can withstand the applied load combination by its structural overall resistance. This is quite easy if the system consists only of one material (such as pure steel structures). If it comes to systems with different structural partners such as reinforced concrete structures (different statistic material scattering properties of concrete and reinforcement) it is no longer quite easy to say which overall global safety has to be validated to meet the required failure probability according to Eurocode 0. In this contribution, different methods of nonlinear global safety formats are shown and discussed with special emphasis on the ECOV method (Červenka 2008) and are applied to a natural draft cooling tower.

1 INTRODUCTION

Since the beginning of the 1990s, the Eurocodes have been introduced into structural design for the first time. Since the middle of the year 2012, the 2nd generation of Eurocodes has been established mandatory substituting national design codes in Germany. The main aim within the probabilistic approach of the new Eurocode generation is to ensure a failure probability of $p_f = 10^{-6}$ within one year which correlates to the reliability index $\beta = 4.7$ for the ultimate limit state in class RC2.

However, in practical structural design the check of failure probabilities is not a standard method to validate the safety level of the structure. Instead, the well-known semi-probabilistic safety concept is used which is based on partial safety factors. Hereby, all characteristic values (both loads and material properties) are multiplied / divided by partial safety factors leading to the design equation of linear elastic design for the persistent or transient situation:

$$E_d = E\{\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,j}\} \quad (1)$$

If the complete stochastic failure probability field is known e.g. as depicted in Figure 1, design values of loading and material properties can be derived which are corresponding to the design point P. The distance from the center (mean values) to the design point is identified as safety index β, the corresponding area above the curve is identified as failure probability, for standard ULS design β = 4,7 and $p_f = 10^{-6}$ (one year) in reliability class RC 2.

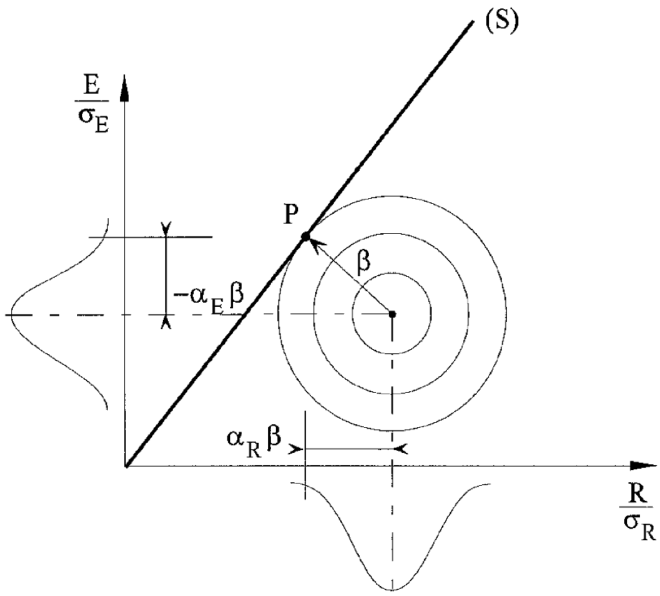


Figure 1: Probability distribution, design point and failure probability (2D)

For different probability distributions, the design values can be obtained by the following relation from the given mean value μ and standard deviation σ while α is taken as 0.8 in general.

Normal distribution $R_d = \mu - \alpha\,\beta\,\sigma$ (2)

Lognormal distribution for $v = \sigma/\mu < 0.2$ $R_d = \mu \cdot e^{-\alpha\,\beta\,\sigma}$ (3)

2 SAFETY FORMATS IN NON-LINEAR ANALYSIS OF REINFORCED CONCRETE

2.1 *General*

Reinforced concrete consists of two partners: Concrete and embedded reinforcing bars in bond. Compressive forces are assigned to concrete and tensile forces are assigned to reinforcement in general. Due to different material property scattering, different material partial safety factors apply: concrete $\gamma_c = 1.50$ and reinforcement $\gamma_s = 1.15$ as defined in EN1992-1-1.

Assuming realistic probabilistic distributions (e.g. according to Fischer 2010), partial safety factors can be computed by the following relations:

Reinforcing steel	B 500	$f_{yk} = 500$ MN/m²	$v = \sigma/\mu = 0.054$
		$f_{ym} = 500 \cdot e^{1.645 \cdot 0.054} \approx 550$ MN/m²	
		$f_{yd} = 550 \cdot e^{-0.8 \cdot 4.7 \cdot 0.054} \approx 448$ MN/m²	
		$\gamma_s = 500/448 = 1.11 \approx 1.15$	
Concrete	C 20/25	$f_{ck} = 20$ MN/m²	$v = \sigma/\mu = 0.20$
		$f_{cm} = 20 \cdot e^{1.645 \cdot 0.20} \approx 27.8$ MN/m² ≈ 28 MN/m²	
		$f_{cd} = 27.8 \cdot e^{-0.8 \cdot 4.7 \cdot 0.20} \approx 13.1$ MN/m² (without factor α_{cc})	
		$\gamma_c = 20.0/13.1 = 1.53 \approx 1.50$	

2.2 *Safety concept of DIN EN 1992-1-1/NA*

Eurocode 2 (EN1992-1-1) allows the application of nonlinear methods for structural analysis in general. However, details about how to perform the analysis are given in the National Appendix, which can be different for every country participating in the Eurocode program. The main problem can be seen in the definition of the global safety factor since concrete and reinforcement have different material scattering as already discussed. In Germany, the following γ_R-safety concept applies where material properties are modified in order to provide a global safety $\gamma_R = 1.30$ for the persistent or transient situation:

$$R_d = R(f_{cR};\, f_{yR}) / \gamma_R \tag{4}$$

This means, that the resistance is calculated within a nonlinear analysis using modified material properties. This global resistance must be greater than $\gamma_R = 1.30$ times loading. The following material properties apply within this scope: $f_{yR}/f_{yk} = \gamma_R/\gamma_s = 1.30/1.15 = 1.13 \approx 1.10$; $f_{yR} = 1.10\ f_{yk}$; $f_{cR}/f_{ck} = \gamma_R/\gamma_c = 1.30/1.50 \approx 0.85$; $f_{cR} = 0.85\ \alpha_{cc}\ f_{ck}$.

This concept works fine for beam structures with main action bending. However, it might become problematic in the authors opinion as soon as the order of failure or the deformation behavior will influence the structural response of highly statically indeterminate structures with biaxial combined membrane and bending action such as cooling tower shells.

2.3 *Safety concept of VGB BTR 2010 (including Appendix C)*

The VGB BTR 2010 is based on EN1992-1-1. In Appendix C which is currently in working progress, special hints for the use of non-linear methods are given. For ultimate load design, two concepts are proposed:

• Concept 1 – γ_s-concept with the use of characteristic material properties

Material properties

Compressive strength concrete	$f_{ck} = \alpha_{cc}\ f_{ck}$	
Tensile strength concrete	$f_{ctk;0.05} = \alpha_{ct}\ 0.70\ f_{ctm}$	for $f_{ck} \leq 50$ MN/m²
Young's modulus concrete	$E_{cm} = 22\ (f_{cm}/10)^{0.3}$	for $f_{ck} \leq 50$ MN/m²
Yield strength steel	f_{yk}	

Design combination with wind as main action $\gamma_G \cdot G + \lambda \cdot \gamma_Q \cdot W$
Design combination with wind not as main action $\gamma_G \cdot G + \lambda \cdot (\gamma_Q \cdot \psi_0 \cdot W + \gamma_Q \cdot T)$

A global safety of $\lambda \geq \gamma_s = 1.15$ has to be checked, while additional considerations concerning failure of concrete have to be carried out due to partial safety factor $\gamma_c = 1.50 > \gamma_s = 1.15$.

• Concept 2 – γ_R-concept adapted from DIN EN 1992-1-1/NA

Material properties
Compressive strength concrete $f_{cR} = 0.85\ \alpha_{cc}\ f_{ck}$
Tensile strength concrete $f_{ctR} = 0.85\ \alpha_{ct}\ f_{ctm}$
Young's modulus concrete $E_{cR} = 22\ ((f_{cR} + 8)/10)^{0.3}$
Yield strength steel $f_{yR} = 1.10\ f_{yk}$

Design combination with wind as main action $\lambda \cdot (\gamma_G \cdot G + \gamma_Q \cdot W)$
Design combination with wind not as main action $\lambda \cdot (\gamma_G \cdot G + \gamma_Q \cdot \psi_0 \cdot W + \gamma_Q \cdot T)$

Two main problems can be identified with the presented concepts which are currently proposed in VGB BTR 2010 Appendix C: In Concept 1, check of concrete is not included. Required safety might be higher up to $\lambda = \gamma_c = 1.50$. In concept 2, modified material properties are used which do not present reality, especially for concrete. Further, the assumption of tensile strength might not be conservative and the actual safety level in terms of failure probability cannot be estimated from these concepts.

2.4 *Safety format ECOV*

The ECOV method is presented by Červenka for general structures. It's advantage is that both mean and characteristic material properties are used in the analysis and from the structural responses a failure probability can be derived. The main drawback is that two nonlinear structural analyses have to be conducted for each load combination.
The main idea behind this concept is that structural resistance for two situations is calculated:

Mean values of material $R_m = R(f_{cm};\ f_{ctm},\ E_{cm},\ f_{ym})$ (5)

Characteristic values of material $R_k = R(f_{ck};\ f_{ctk},\ E_{ck},\ f_{yk})$ (6)

With the knowledge of resistance R_m and R_k, a probability distribution is assumed following the Log-Normal distribution which is commonly used for the description of engineering materials. From this distribution, the coefficient of variation v is estimated (giving the name of this method ECOV):

$$v = \frac{1}{1.65} \ln\left(\frac{R_m}{R_k}\right) \tag{7}$$

From the coefficient of variation, the required safety factor γ_{Glob} is computed using the required safety coefficient $\beta = 4.7$ and $\alpha = 0.80$ as described in chapter 1:

$$\gamma_{Glob} = e^{\alpha \cdot \beta \cdot v} = e^{3.76 \cdot v} \tag{8}$$

2.5 *Full probabilistic analysis*

Full probabilistic method requires a variety of results using different combinations of stochastic variables (both loads and materials). From this set of results (much more than 2 sets as used in ECOV method), the probability distribution together with the design point β is calculated. Hereby, Monte-Carlo simulation has to be mentioned. However, for this type of application full probabilistic analysis is far away from practical use.

3 APPLICATION STUDY NATURAL DRAFT COOLING TOWER

3.1 *Geometry*

Within an application study of a natural draft cooling tower, the application of ECOV method is used. The geometry, shell thickness and installed reinforcement are depicted in Figure *2* and Figure 3. The geometry is described by one hyperbolic function for the complete height. On Top, the tower is stiffened by a ring beam.

$$R(\theta^2) = 10.945 + 18.49\sqrt{1 + \left(\frac{\theta^2 - 109.14}{58.268}\right)^2} \tag{9}$$

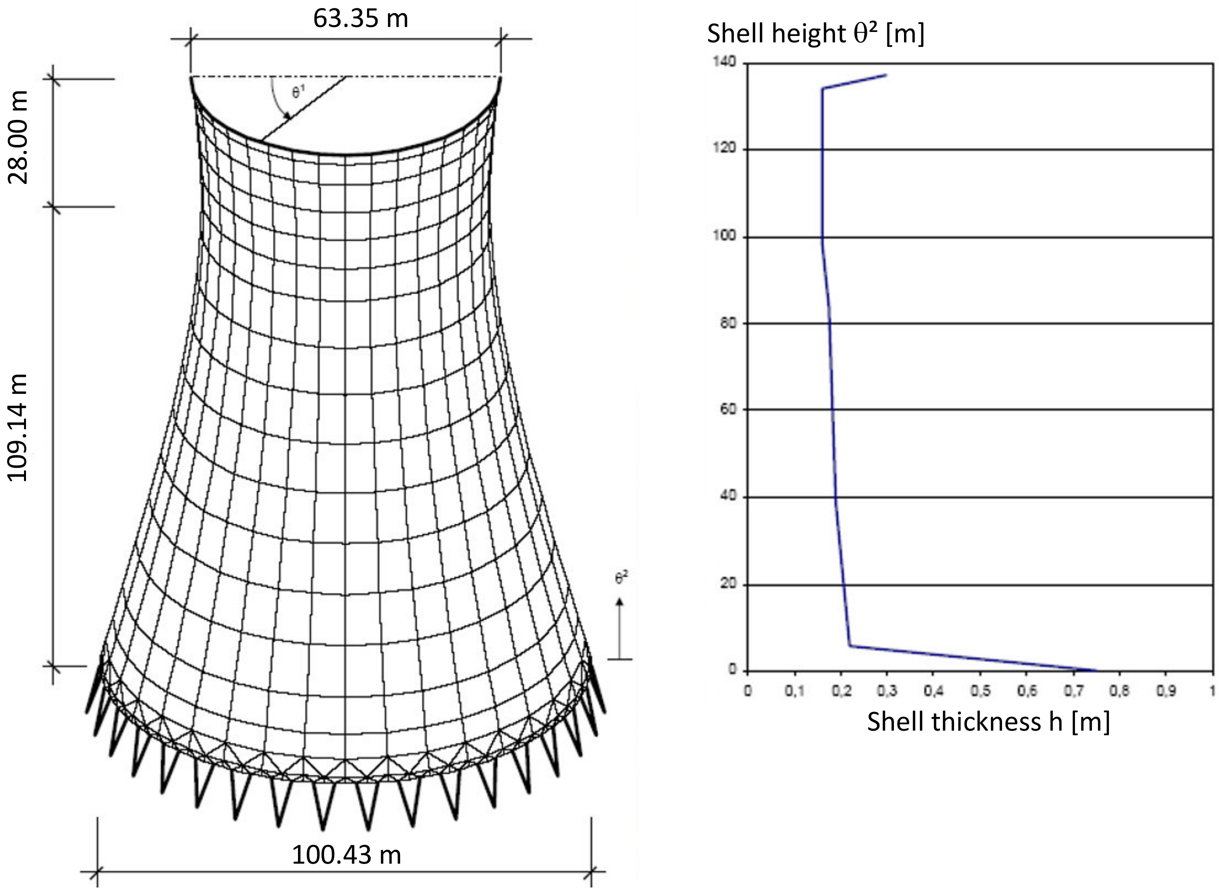
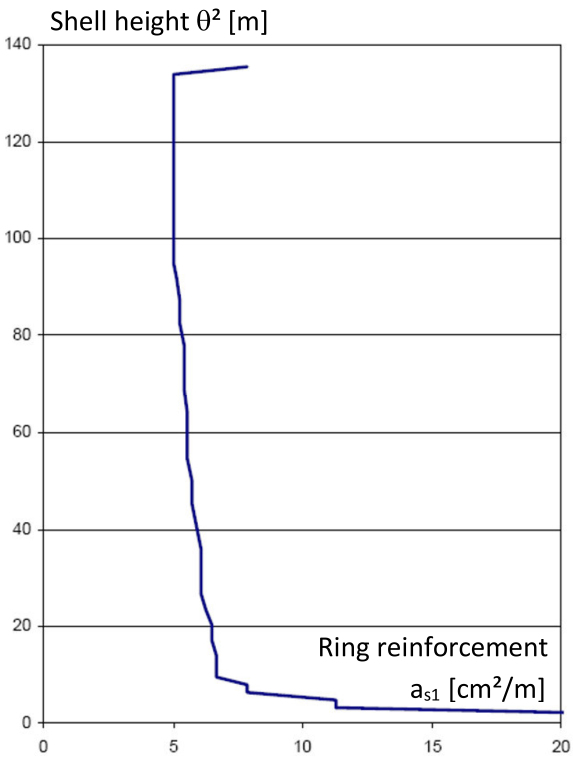


Figure 2: Geometry and wall thickness of analyzed cooling tower



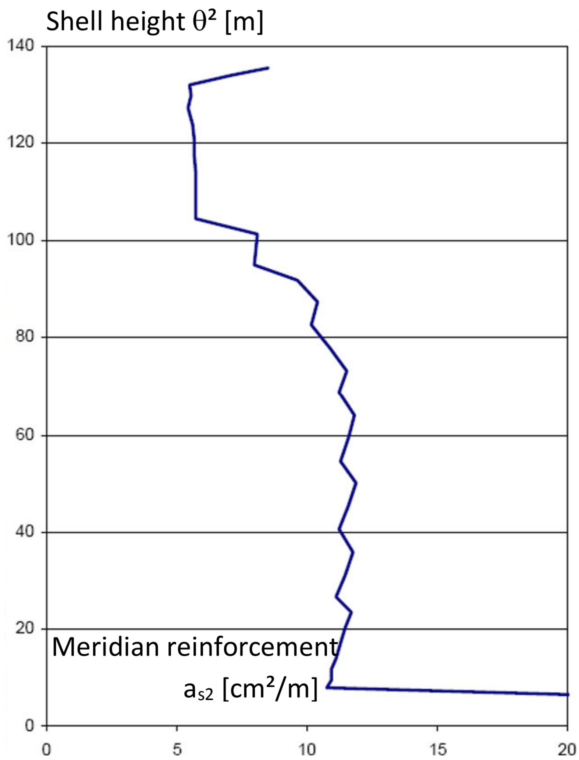


Figure 3: Installed reinforcement

3.2 *Material properties*

The following material properties are assumed within this study:

a) Design values C 20/25, B 500
   $f_c = 0.85 \cdot 20/1.50 = 11.3$ MN/m
   $E_c = 1.05 \cdot 22000\ (11.3/10)^{0.3} = 24000$ MN/m²
   $f_{ct} = 0.85 \cdot 0.7 \cdot 0.3 \cdot 20^{2/3}/1.5 = 0.85$ MN/m²
   $f_y = 500/1.15 = 435$ MN/m²

b) Characteristic values C 20/25, B 500
   $f_c = 0.85 \cdot 20 = 17$ MN/m
   $E_c = 1.05 \cdot 22000\ (2.0/10)^{0.3} = 27100$ MN/m²
   $f_{ct} = 0.85 \cdot 0.7 \cdot 0.3 \cdot 20^{2/3} = 1.3$ MN/m²
   $f_y = 500$ MN/m²

c) Mean values C 20/25, B 500
   $f_c = 0.85 \cdot 28 = 23.8$ MN/m
   $E_c = 1.05 \cdot 22000\ (23.8/10)^{0.3} = 30000$ MN/m²
   $f_{ct} = 0.85 \cdot 0.3 \cdot 20^{2/3} = 1.87$ MN/m²
   $f_y = 550$ MN/m²

3.3 *Loading*

While dead weight of the shell is assumed with $\gamma = 25.2$ kN/m² considering the actual shell rib geometry, the wind loading is described by the following relation with reference to VGB BTR for curve K1.1 regarding $c_{pe}(\theta^1)$ for circumferential distribution.

$$w(\theta^1, Z) = c_{pe}(\theta^1) \cdot 0.90 \cdot (Z/10)^{0.22} \quad [kN/m^2] \tag{10}$$

Within the non-linear pushover analyses, the following load combinations are analyzed. Hereby, load factor λ is either applied on the full combination or only on wind in situations where dead weight is acting favorable.

1) $\lambda \cdot (G + 1.50\ W)$
2) $G + \lambda \cdot 1.50\ W$
3) $\lambda \cdot (1.35\ G + 1.50\ W)$ not further presented

3.4 *Safety considerations*

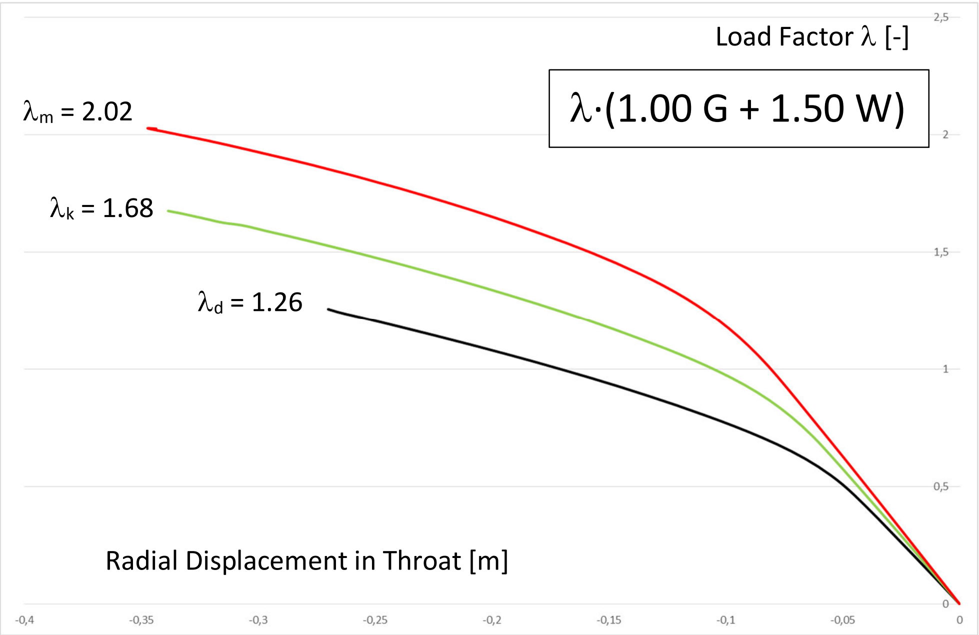


Figure 4: Results for $\lambda \cdot (G + 1.50 \cdot W)$

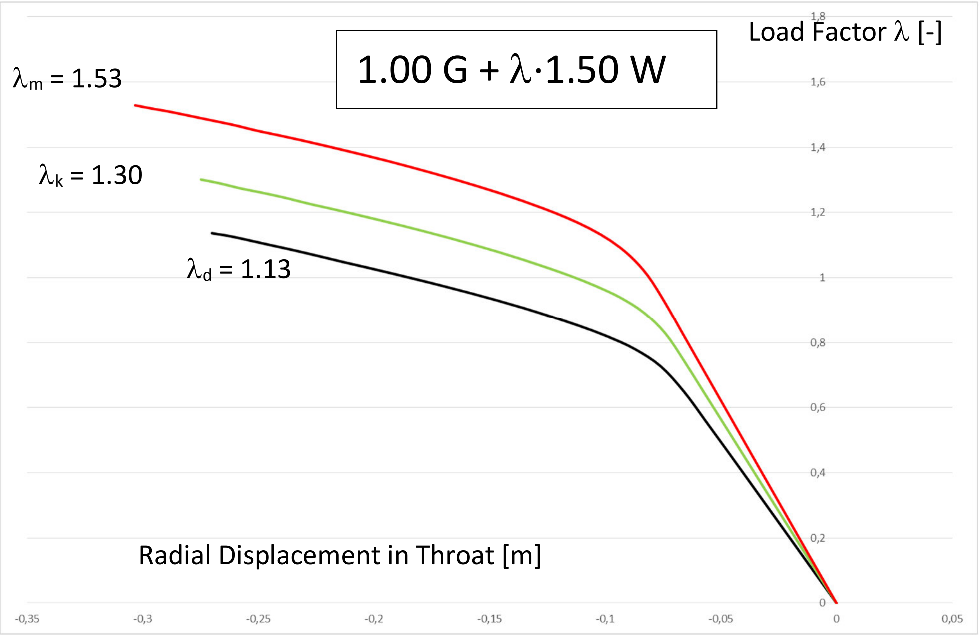


Figure 5: Results for $G + \lambda \cdot 1.50 \cdot W$

The required load factor $\lambda_d \geq 1.00$ is met for all analyzed load combinations. However, the material safety factor is already included in material properties, so that the material properties used in nonlinear analysis do not represent reality and the pushover behavior with the assumption of these values has to be questioned.

Using ECOV method, the safety check will result in the following:

Combination 1	$\lambda\cdot(G + 1.50\ W)$	$\lambda_k = 1.68 \quad \lambda_m = 2.02$ $v = 1/1.65\ \ln(2.02/1.68) = 0.11$ $\gamma_{Glob} = e^{3.76\cdot 0.11} = 1.51 < \lambda_m = 2.02$ ok
Combination 2	$G + \lambda\cdot 1.50\ W$	$\lambda_k = 1.30 \quad \lambda_m = 1.53$ $v = 1/1.65\ \ln(1.53/1.30) = 0.10$ $\gamma_{Glob} = e^{3.76\cdot 0.10} = 1.45 < \lambda_m = 1.53$ ok
Combination 3	$\lambda\cdot(1.35\ G + 1.50\ W)$	$\lambda_k = 2.06 \quad \lambda_m = 2.51$ $v = 1/1.65\ \ln(2.51/2.06) = 0.12$ $\gamma_{Glob} = e^{3.76\cdot 0.12} = 1.57 < \lambda_m = 2.51$ ok

The safety factor γ_{Glob}, however, relates to the mean values of resistance. In all three cases, the safety level could be met. Decisive for design is the combination $G + \lambda\cdot 1.50\ W$. From the nonlinear analysis with design material properties, a safety factor of 1.13 could be identified. In comparison with the ECOV method, a safety level of 1.53/1.45 = 1.06 could be identified.

4 SUMMARY AND CONCLUSIONS

In this paper, a different safety format has been presented for non-linear structural analysis with adaptation to natural draft cooling towers. This method is based on the estimation of the coefficient of variation and therefore called ECOV method. It has been recently published and proposed by Červenka. The main advantages of this method are that mean material values and characteristic material properties are used for the analysis and no fictive modifications of material properties must be done in order to adapt the different partial safety factors of the bond partners in reinforced concrete. Further, a direct link to the failure probability is obtained. The main drawback can be seen in the fact, that two non-linear analyses have to be conducted for each load combination. However, it is recommended to include this design concept into the recommendations of the next edition of VGB BTR R 610 due to its more general approach.

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