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# Partial factors for the design resistance of composite beams in bending

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ABSTRACT

This paper presents the results from a reliability analysis of the resistance of composite beams in sagging bending designed according to Eurocode 4. Using the EN 1990 methodology, the partial factors $\gamma_M$ for the structural steel, concrete and shear connection were evaluated. The present study extends earlier work by considering geometrical tolerances given within the published European product and execution standards, which were unavailable during the original calibration of Eurocode 4. Furthermore, recently reported European production data on the yield strength of structural steel is also included. The analyses consider test data from 164 beams with full shear connection, partial shear connection, ductile connectors, non-ductile connectors and beams with high strength steel, which ar?e supplemented with over 3 million simulations. It was found in the present work that the current recommended values for $\gamma_M$ were only justified for beams with full shear connection. For beams with partial shear connection, the calculated values of $\gamma_M$ were larger than recommended because the partial factor associated with the uncertainty of the resistance model varied considerably. To remedy this situation, conversion factors that are a function of the overall composite beam depth are proposed which, when applied to the design models, justifies lower partial factors than that currently recommended by Eurocode 4.


## 1. Introduction

The structural Eurocodes have been developed based on the partial factor method applied in conjunction with the concept of limit states (ultimate, serviceability or fatigue). According to EN 1993-1-1 [1] (Eurocode 3), for structural steel cross-sections that are not influenced by buckling at the ultimate limit state, the partial factor $\gamma_{M0}$ should be applied to the characteristic values of the materials that are contained within the design equation for resistance; whereas, for cases when the resistance of members is influenced by buckling, the entire design equation is divided by the partial factor $\gamma_{M1}$. For structural concrete, the partial factor given in EN 1992-1-1 [2] (Eurocode 2) for persistent and transient design situations is $\gamma_C$.

To encourage harmonisation across borders and to increase the usability of the Eurocodes, it was considered important that the recommended values in EN 1994-1-1 [3] (Eurocode 4) were identical to those given in Eurocode 2 and Eurocode 3, but composite beams in sagging bending consist of three different materials: $\gamma_{M0}$ for structural steel and steel sheeting; $\gamma_C$ for structural concrete; and $\gamma_V$ for shear connectors. Johnson and Huang [4] were responsible for the calibration of the pre-standard ENV 1994-1-1 [5] where, at the time, $\gamma_{M0} = 1.10$, $\gamma_C = 1.50$ and $\gamma_V = 1.25$. However, in the publication of the final version of Eurocode 3, the recommended value was lowered to $\gamma_{M0} = 1.00$, yet the effect of this change on the final version of Eurocode 4 was never considered. Moreover, as not all of the product and execution standards had been published at the time, Johnson and Huang based their analyses on coefficients of variation that had historically been used to calibrate design standards in the Netherlands [6].

More recently, Mujagić and Easterling [7] conducted a reliability study of composite beams in sagging bending to justify the resistance factor $\phi$ in the 1999 and 2005 AISC Specification [8,9]. However, the results from this work cannot be used directly in Eurocode 4 owing to the fact the North American format combines all of the material and resistance model uncertainties into the factor $\phi$. Also, the AISC Specification only requires that the degree of shear connection $\eta > 50\%$, with no direct account being made of the slip capacity of the shear connectors, so the results from Mujagić and Easterling include beams that are deemed to be non-ductile according to Eurocode 4. Finally, the factor $\phi$ is determined from a coupled reliability analysis that considers both flexural strength and flexural demand from the applied loads [10], whereas the Eurocodes uncouple the actions and resistances so that the partial factors for each can be considered separately.

In this paper, the partial factors for the sagging bending resistance of composite beams designed to Eurocode 4 are reconsidered, using a wider range of test data than was used in the original calibration by Johnson and Huang together with the work of Mujagić and Easterling. In addition, tolerances given within the published European product and execution standards, together with recent production data on the yield strength of steel, are implemented within the analyses. Beams with full shear connection, partial shear connection and non-linear

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resistance are considered (when the shear connection is deemed to be non-ductile). Furthermore, the study is extended to beams using high strength steel with a nominal yield strength $f_y \geq 460$ MPa.

## 2. Overview of partial factor design and reliability analysis

In probability-based design, the probability of failure $P_f$ is the basic reliability measure that is used in international Standards such as ISO 2394 [11]. An alternative measure that is used in the head code to the Eurocode suite EN 1990 [12] is the reliability index $\beta$, which is related to the probability of failure $P_f$ by:

$$P_f = \Phi(-\beta) \tag{1}$$

where $\Phi$ is the cumulative distribution function of the standardised normal distribution and $\beta$ is the reliability index.

For ultimate limit state considerations, the target reliability index given in EN 1990 for a 50-year reference period is $\beta = 3.8$ for Reliability Class 2 (RC2) structural members. According to EN 1990, RC2 members are appropriate for a Consequence Class 2 (CC2) structure where there is a '*medium* consequence for loss of human life, economic, social or environmental consequences *considerable*'; examples of CC2 structures are residential and office buildings, public buildings where consequences of failure are medium (this target value for the reliability index is consistent with that recommended by ISO 2394 for great consequences of failure where the relative costs of safety measures are moderate). Design values of resistances are defined such that the probability of having a more unfavourable value is as follows:

$$P(R \leq R_d) = \Phi(-\alpha_R \beta) \tag{2}$$

where $\alpha_R$ is the FORM (first order reliability method) sensitivity factor for resistance.

Both ISO 2394 and EN 1990 give $\alpha_R = 0.8$ for a dominating resistance parameter (defined in EN 1990 when $0.16 < \sigma_E/\sigma_R < 7.6$, where $\sigma_E$ and $\sigma_R$ are the standard deviations of the action effect and resistance, respectively). Therefore, according to EN 1990, the design value for resistance corresponds to the product $\alpha_R\beta = 0.8 \times 3.8 = 3.04$ (equivalent to a probability of the actual resistance falling below the design resistance of 1 in $845 = P_f = 0.0012$). The remaining safety is achieved in the specification of the actions. The design resistance is defined in EN 1990 as:

$$R_d = \frac{1}{\gamma_{Rd}} R\left\{X_{d,i}; a_d\right\} = \frac{1}{\gamma_{Rd}} R\left\{\eta_i \frac{X_{k,i}}{\gamma_{m,i}}; a_d\right\} \qquad i \geq 1 \tag{3}$$

where $\gamma_{Rd}$ is the partial factor associated with the uncertainty of the resistance model (according to ISO 2394 $\gamma_{Rd}$ should, in general, be $\gamma_{Rd} \geq 1.0$), $X_{d,i}$ is the design value of material property $i$, $\eta_i$ is the design value of the conversion factor for property $i$ (which converts properties obtained from test specimens to properties corresponding to the assumptions made in calculation models, such as size effects, time effects, etc.), $X_{k,i}$ is the characteristic value of material property $i$, $\gamma_{m,i}$ is the partial factor for material property $i$ and $a_d$ is the design value for geometrical data (which is normally taken as the nominal geometrical value $a_{nom}$, unless the deviations in the geometrical data have a significant effect on the reliability of the structure, such as imperfections in a buckling analysis).

The partial factors in Eq. (3) can be simplified by the following definition, which enables a calibration to be undertaken for any structural element composed of more than one material:

$$\gamma_{M,i} = \gamma_{Rd}\gamma_{m,i} \qquad i \geq 1. \tag{4}$$

## 3. Evaluation of partial factor $\gamma_M$ from testing according to EN 1990

A method for evaluating the design resistance of steel structures from tests was developed by Bijlaard et al. [13], which has subsequently been implemented within EN 1990, Annex D as the standard evaluation procedure for all materials. A design model is defined for the theoretical resistance of the member or component under consideration, which includes all the relevant basic variables $\underline{X}$ that control the resistance at the limit state $g_{rt}(\underline{X})$. The basic variables are considered as random variables, with negligible correlation between them, and have a log-normal distribution (which is desirable as it falls to zero at the origin, so there are no negative resistances).

The theoretical resistance is expected to differ from the true experimental resistance by a correction factor $b_i$, which defined as:

$$b_i = \frac{r_{ei}}{r_{ti}} \tag{5}$$

where $r_{ei}$ is the experimental resistance for specimen $i$ and $r_{ti}$ is the theoretical resistance predicted from the design model using a set of mean measured basic variables that are included in a report from a laboratory test on specimen $i$.

The coefficient of variation $V_r$ is obtained from two sources: the coefficient of variation of the error term $V_\delta$, from a consideration of the scatter of $b_i$; and the coefficient of variation of the theoretical resistance $V_{rt}$, from uncertainties in the basic variables. According to EN 1990, $V_r$ may be approximated to be:

$$V_r = \sqrt{V_\delta^2 + V_{rt}^2} \tag{6}$$

and $V_{rt}$ may be determined from:

$$V_{rt}^2 = \frac{VAR[g_{rt}(\underline{X})]}{g_{rt}^2(\underline{X}_m)} \cong \frac{1}{g_{rt}^2(\underline{X}_m)} \sum_{i=1}^{j} \left(\frac{\partial g_{rt}}{\partial X_i}\sigma_i\right)^2 \tag{7}$$

where $\sigma_i$ is the standard deviation for basic variable $i$ and $\underline{X}_m$ is the mean value of the basic variables.

The characteristic or design value is based on the prediction method in EN 1990, which is a procedure for estimating a population's fractile from an available sample of limited size $n$. If the coefficient of variation of the population is known (defined as "$V_X$ known" in EN 1990), the fractile factor is given by:

$$k_n = -u_p(1/n+1)^{1/2} \tag{8}$$

where $u_p$ is the $p$ fractile of the standardised normal distribution and $n$ is the size of the population.

Alternatively, if the coefficient of variation of the population is unknown (defined as "$V_X$ unknown" in EN 1990), the fractile factor is given by:

$$k_n = -t_p(1/n+1)^{1/2} \tag{9}$$

where $t_p$ is the $p$ fractile of the known Student $t$-distribution (with $v = n - 1$ degrees of freedom) and $n$ is the size of the population.

According to EN 1990, it is preferable to assume that the coefficient of variation of the population is "$V_X$ known" (i.e. Eq. (8)). Therefore, $V_X$ known was assumed in the analyses presented in this paper.

The characteristic value of the resistance $R_k$ is given by:

$$R_k = b g_{rt}(\underline{X}_m) \exp\left(-k_\infty \alpha_{rt} Q_{rt} - k_n \alpha_\delta Q_\delta - 0.5Q^2\right) \tag{10}$$

where $b$ is the mean value of the correction factor (taken as the slope of a least squares fit line through $r_{ei}$ and $r_{ti}$), $k_n$ is the characteristic fractile factor for size of population $n$ (from either Eqs. (8) or (9) with