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RELIABILITY ASSESSMENT OF DESIGN RULES FOR STAINLESS STEEL STRUCTURES

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Abstract. *This paper presents a re-evaluation of the current partial resistance factors recommended in EN 1993-1-4 for the design of stainless steel elements. Material data from key stainless steel producers were collected and carefully analysed, and representative values of the over-strength and the coefficient of variation (COV) of the material yield strength and ultimate tensile strength, necessary for performing reliability analysis, were established. The EN 1990 Annex D First Order Reliability Method (FORM) was applied to a substantial pool of experimental results. At the cross-section level, stub column and in-plane bending test results were used to assess the γ_{M0} partial resistance factor. At the member level, flexural buckling and lateral-torsional buckling test results were used to evaluate the γ_{M1} partial resistance factor. It is revealed that the current recommended partial resistance factors in EN 1993-1-4 ($\gamma_{M0} = \gamma_{M1} = 1.1$) cannot generally be reduced, and in some cases, modified design resistance equations are required, if the current safety factors are to be maintained.*

1 INTRODUCTION

Three partial safety factors, γ_{M0}, used in cross-section design checks, γ_{M1}, employed in member instability design checks, and γ_{M2}, used in expressions for determining the resistance of cross-sections in tension and the resistance of bolted and welded connections, are used in EN 1993-1-4 [1] for the design of stainless steel structural members. The partial resistance factors allow for uncertainties in the material properties, the geometric properties and the accuracy of the design resistance functions, and their values are obtained through calibration of the codified design resistance equations, using reliability methods to achieve a certain target reliability requirement. The recommended values in EN 1993-1-4 [1] are: $\gamma_{M0} = \gamma_{M1} = 1.1$ and $\gamma_{M2} = 1.25$. Since the establishment of the EN 1993-1-4, a substantial pool of experimental results and statistical material and geometric data have been generated. The objective of this study is therefore to re-evaluate these recommended partial factors in light of this information with a focus on cross-section and member resistances (i.e. γ_{M0} and γ_{M1}). The theoretical background of the reliability method adopted in the Eurocodes, as outlined in EN 1990 [2], is briefly described. The statistical data on material and geometric properties of structural stainless steel sections from the literature and stainless steel producers are then presented. Finally, reliability assessments of the EN 1993-1-4 [1] design resistance equations are carried out, covering: cross-sections in compression, flexural buckling, in-plane bending and lateral-torsional buckling. Note that a more comprehensive account of this investigation is reported in [3].

2 METHODOLOGY FOR THE STATISTICAL EVALUATION OF RESISTANCE MODELS

2.1 Theoretical background

The safety assessment of the resistance functions employed in Eurocode 3 is based on a statistical evaluation of relevant experimental data, carried out within a probabilistic reliability theory framework, leading to the determination of the γ_M values. Equation (1) presents the probability of failure, P_f, i.e. the probability that the resistance (R) minus the action effect (E) is less than zero, in terms of the cumulative distribution function of the standardised normal distribution ϕ evaluated for a total reliability index β. The total reliability index β, which is selected based on a series of consequence classes (CC) is directly related to the reliability classes (RC), as defined in Annex C of EN 1990 [2]. A value of $\beta = 3.8$ has been adopted in the analyses performed in this paper, which corresponds to typical building structures will fall into reliability class RC2 with a reference design life of 50. To calibrate the codified design resistance functions, a semi-probabilistic approach, where the variabilities of the load effects and resistance functions are assessed separately has been used in EN 1990 [2], through the use of FORM sensitivity factors α_E and α_R, resulting in Equations (2) and (3) for the action effect and resistance, respectively, where E_d is the design action effect and R_d is the design resistance. Hence, to establish the partial safety factor for a new design procedure, only Equation (3) needs to be considered.

$$P_f = P[(R-E)\le 0] = \phi(-\beta) \tag{1}$$

$$P(E > E_d) = \phi(\alpha_E \beta) \tag{2}$$

$$P(R \le R_d) = \phi(\alpha_R \beta) \tag{3}$$

The sensitivity factors may be approximately taken as $\alpha_E = -0.7$ and $\alpha_R = +0.8$, provided that the ratio of the standard deviation of the action effect σ_E and resistance σ_R is such that $0.16 \le \sigma_E/\sigma_R \le 7.6$ [2]. This means that for reliability class RC2, the probability of the resistance of structural components falling below the design resistance is as given in Equation (4). The partial resistance factor γ_M, given in Equation (5) is defined as the ratio of the nominal resistance value r_n, determined from the design resistance equation under consideration, using the nominal geometric and material properties, and the design resistance value r_d, determined from the reliability analysis procedures using the values of basic variables measured during testing.

$$P(R \le R_d) = \phi(-0.8 \times 3.8) \approx 0.001 \tag{4}$$

$$\gamma_M = \frac{r_n}{r_d} \tag{5}$$

2.2 EN 1990 Annex D method

In Annex D of EN 1990 [2], a set of application rules for obtaining the design values for a resistance function through a statistical evaluation of experimental data is provided. The method begins by comparing the theoretical resistance values $r_{t,i}$ obtained from the resistance function under consideration $g_{rt}(X)$, using the measured material and geometric properties, with the experimental resistance values $r_{e,i}$ from each test, through a plot of $r_{e,i}$ versus $r_{t,i}$ values. An error term $\delta_i = r_{e,i}/br_{t,i}$, is calculated for each $(r_{t,i},r_{e,i})$ data pair, showing the deviation of the experimental resistance values to the mean strength function $r_e = br_t$, where b is the mean value correction factor obtained as the least squares best fit of the slope of the $r_{e,i}$ versus $r_{t,i}$ plot. The coefficient of variation of this error term V_δ is used as a measure of the variabilities associated with the predictions from the resistance function. Considering the logarithmic normal probability distribution of δ_i, the coefficient of variation of the error term is given by Equation (6), where σ_δ^2 is the corresponding variance.

$$V_\delta = \sqrt{\exp\left(\sigma_\delta^2\right)-1} \tag{6}$$

If the scatter of the predictions is too high, i.e. large V_δ values, to give an economical design resistance model, procedures to reduce the scatter are required. The scatter may be reduced by improving the design model to take into account parameters which had previously been ignored, or by modifying the parameters b and V_δ by dividing the total test population into appropriate sub-sets for which the influence of such additional parameters may be considered to be constant. In this study, the test data have been split into sub-sets based on their material grade, as explained in more detail in Section 3. The disadvantage of splitting the test results into sub-sets is that the number of test results in each sub-set can become very small. In order to avoid unreasonably large safety factors as a result of this, Clause D.8.2.2.5 of EN 1990 Annex D [2] allows the use of the total number of tests in the original series for determining the $k_{d,n}$ fractile factor. Hence, in this study the $k_{d,n}$ for each sub-set was based on the total number of tests for all stainless steel grades, for the cross-section shape and failure mode under consideration.

The effect of the variability of the basic variables in the resistance function $g_{rt}(X)$, including material and geometric properties, is also accounted for through their coefficient of variation parameter, V_{rt}. There are two methods of calculating V_{rt}, depending on the level of complexity of the resistance function under consideration. For the case of complex and multi-variable resistance functions, such as the column buckling formula in EN 1993-1-4 [1], V_{rt} may be obtained from Equation (7), where $g_{rt}\left(\underline{X}_m\right)$ is the resistance function evaluated for the mean values of the basic variables and $(\partial g_{rt}/\partial x_i)\sigma_i$ is the partial derivative for the variable X_i multiplied by its respective standard deviation σ_i. Equation (8) is deemed sufficient for resistance functions of simpler form, such as that for the bending resistance of laterally restrained beams, where the coefficient of variation of each of the basic variables V_{Xi} is used directly. The analyses carried out in this paper have made use of both methods as appropriate; this is explained in more detail in Section 4. The coefficients of variation V_{Xi} of the basic variables are generally determined on the basis of prior knowledge, and have been obtained herein using representative data from stainless steel producers as discussed in more detail in Section 3.

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

$$V_{rt}^2 = \sum_{i=1}^{j} V_{Xi}^2 \tag{8}$$

Finally, the design resistance value r_d, leading to the determination of the partial factor γ_M is obtained from Equation (9), which applies in cases of a limited number of test results ($n \leq 100$). In Equation (9), b is the mean value correction factor, $g_{rt}(\underline{X}_m)$ is the design resistance evaluated for the mean values of the basic variables, $k_{d,n}$ is the design fractile factor and $k_{d,\infty}$ is the design fractile factor for n tending to infinity ($k_{d,\infty} = 3.04$). The following parameters: α_{rt} = weighting factor for Q_{rt}, α_δ = weighting factor for Q_δ, Q_{rt}, Q_δ and Q - as defined by Equations (10), (11), (12), (13) and (14), respectively are used to simplify the representation of the calculations.

$$r_d = b g_{rt}(\underline{X}_m) \exp\left(-k_{d,\infty}\alpha_{rt}Q_{rt} - k_{d,n}\alpha_\delta Q_\delta - 0.5Q^2\right) \tag{9}$$

$$\alpha_{rt} = \frac{Q_{rt}}{Q} \tag{10}$$

$$\alpha_\delta = \frac{Q_\delta}{Q} \tag{11}$$

$$Q_{rt} = \sqrt{\ln\left(V_{rt}^2 + 1\right)} \tag{12}$$

$$Q_\delta = \sqrt{\ln\left(V_\delta^2 + 1\right)} \tag{13}$$

$$Q = \sqrt{\ln\left(V_r^2 + 1\right)}, \text{with}\, V_r^2 = V_\delta^2 + V_{rt}^2 \tag{14}$$

3 STATISTICAL DATA ON MATERIAL AND GEOMETRIC PARAMETERS

3.1 Statistical data on yield strength

Mean values and standard deviations for the yield strength, taken as the 0.2% proof stress, of different stainless steel grades were collected from a number of major European stainless steel producers and from the literature [5–9]. Where a number of grades were reported, average values for each stainless steel type austenitic, duplex and ferritic - were determined. A summary of the results is presented in Table 1, where the data within each stainless steel type have been grouped based on the product type - cold-rolled coil/sheet (C), hot-rolled coil/sheet (H) and hot-rolled plate (P). Since the data from the stainless steel producers were provided on a confidential basis, the identity of the producers have not been stated and the source is simply indicated as Producer.

The ratio of mean to minimum specified yield strength $f_{y,mean}/f_{y,min}$ and the coefficient of variation (COV) of the mean yield strength are also provided in Table 1. The minimum yield strength values were obtained from EN 10088-4 [10]. One of the assumptions made in the reliability analysis procedures set out in EN 1990 Annex D is that the minimum (nominal) yield strength, $f_{y,min}$, is a characteristic value and should therefore correspond to the 95% confidence limit. The characteristic yield strengths $f_{y,k}$ corresponding to each set of $f_{y,mean}$ and standard deviation σ data have been evaluated, and the ratios of $f_{y,k}/f_{y,min}$ are reported in Table 1. The fact that the values of $f_{y,k}/f_{y,min}$ are greater than unity indicates that the assumption that nominal yield strength is a characteristic value is conservative; this has also been found for the case of carbon steel [11]. Benefit may be derived from the margin between the nominal and characteristic strength in the reliability analysis, thorough the use of the over-strength parameter $f_{y,mean}/f_{y,min}$, where $f_{y,mean}$ is the mean value produced by stainless steel manufacturers and $f_{y,min}$ is the minimum specified value in EN 10088-4 [10].

During the initial calibration of the EN 1993-1-4 [1] design rules, the over-strength factor for the material yield strength $f_{y,mean}/f_{y,min}$ was taken as 1.33 with a COV value of 0.066 for all stainless steels [12]. Analyses of the results in this study have shown that, in fact, these statistical parameters vary between the different stainless steel types, and their effect needs to be allowed for in the reliability analysis by dividing the structural performance data into sub-sets based on their material grade. From the assembled data in Table 1, on average, the austenitic grades exhibit the highest ratio of $f_{y,mean}/f_{y,min}$ of 1.40, the lowest of 1.20 is shown by the duplex grades, and an intermediate value of 1.38 is observed for the ferritic grades. The range of $f_{y,mean}/f_{y,min}$ values for the different stainless steels is 1.34 - 1.54 for the austenitic grades, 1.04 - 1.33 for the duplex grades and 1.21 - 1.51 for the ferritic grades. In the present study, representative but conservative values of over-strength were sought. Hence, based generally on the minimum over-strength values from the different sources (producers), values of 1.3, 1.1 and 1.2 for the austenitic, duplex and ferritic grades, respectively were considered appropriate for use in the reliability analyses.

The coeffcients of variation of the yield strength are plotted against the $f_{y,mean}/f_{y,min}$ ratio for all grades in Figure 1. The data reveals a clear trend, common to all grades, of reducing COV with reducing $f_{y,mean}/f_{y,min}$. This would be anticipated since, as the $f_{y,mean}/f_{y,min}$ ratio approaches unity, tighter controls would be required by the manufacturers to ensure that the material satisfies the minimum requirements. The linear regression relationship between the $f_{y,mean}/f_{y,min}$ ratio and COV values, shown in Figure 1, was used to obtain COV values corresponding to the adopted over-strength factors. The COV values were equal to 0.060, 0.030 and 0.045 for the austenitic, duplex and ferritic grades, respectively.

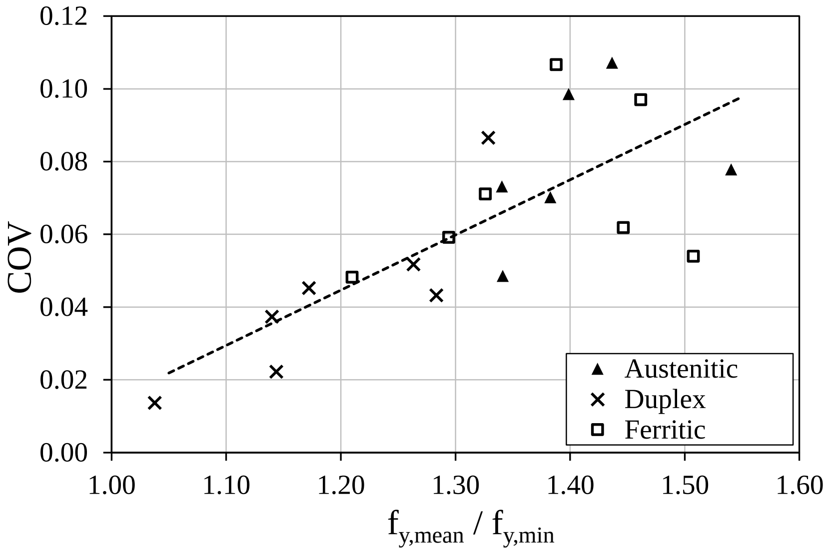

Figure 1: Relationship between $f_{y,mean}/f_{y,min}$ and COV

3.2 Statistical data on ultimate tensile strength

A similar analysis to that described above was carried out for the ultimate tensile strength f_u of stainless steel, and the results are summarised in Table 2. The over-strength factor for the ultimate tensile strength $f_{u,mean}/f_{u,min}$ fell into a tight range of between 1.06 and 1.23 for all stainless steel grades. Hence, a single over-strength value $f_{u,mean}/f_{u,min}$, common to all stainless steel grades of 1.1, which is close to the lower end of this range, was deemed appropriate for use in reliability analyses. Also, owing to the relatively narrow band of $f_{u,mean}/f_{u,min}$, no clear correlation between the over-strength and the associated COV, as had been seen for the case of the yield strength, could be established. Therefore, considering the range of COV values obtained from the individual sources for the austenitic and duplex stainless steel grades, 0.017-0.034 and 0.010-0.038, respectively, a common COV value towards the upper end of these ranges of 0.035 is proposed. To allow for the generally larger scatter obtained from the individual sources for the ferritic material, 0.024-0.068, a higher COV value of 0.05 is proposed herein. This value is towards the upper end of the range of COV values from the individual sources and is only exceeded by two data sets, both with $f_{u,mean}/f_{u,min}$ = 1.14, which is higher than the adopted value of 1.1, and would therefore be expected to off-set the effect of the lower COV adopted.

Table 1: Statistical data on material yield strength

Material type	Product type	Source	No. of tests n	Thickness range (mm)	$f_{y,mean}$ (N/mm^2)	σ (N/mm^2)	COV	$f_{y,mean}/f_{y,min}$	$f_{y,k}/f_{y,min}$
Austenitic	C	[5]	2572	2.49-6.35	312	15.2	0.049	1.34	1.24
		Producer	-	-	314	22.9	0.073	1.34	1.19
	H	[7,8]	-	4.0	290	-	-	1.37	-
		Producer	-	-	326	25.3	0.078	1.54	1.35
	P	[6]	>3000	5.0-50	294	20.6	0.070	1.38	1.23
		[7,8]	-	15	283	-	-	1.33	-
		Producer	1368	-	309	33.0	0.107	1.44	1.20
		Producer	-	-	293	28.8	0.099	1.40	1.19
	Average				308			1.40	1.23
Duplex	C	[5]	239	2.49-6.35	586	26.5	0.045	1.17	1.09
		[9]	-	1.0	650	-	-	1.27	-
		Producer	5749	0.4-3.5	631	27.3	0.043	1.28	1.19
		Producer	-	-	610	30.9	0.052	1.26	1.16
		Producer	-	<6.4	550	7.5	0.014	1.04	1.01
	H	[9]	-	4.0	595	-	-	1.27	-
		Producer	-	-	591	49.0	0.087	1.33	1.16
		Producer	-	<10	549	12.2	0.022	1.14	1.10
	P	[6]	>300	5.05-50	524	19.6	0.037	1.14	1.07
		[9]	-	15	505	-	-	1.11	-
		Producer	-	-	520	18.2	0.035	1.19	1.13
	Average				570			1.20	1.12
Ferritic	C	Producer	-	-	331	19.0	0.059	1.29	1.17
		Producer	-	-	349	21.4	0.062	1.45	1.31
		Producer	-	>10	358	19.3	0.054	1.51	1.38
		Producer	438	1.25-2.0	352	16.9	0.048	1.21	1.12
	H	Producer	-	-	354	34.0	0.097	1.46	1.25
		Producer	-	-	371	26.4	0.071	1.33	1.18
	P	Producer	-	-	347	37.0	0.107	1.39	1.16
	Average				352			1.38	1.22

Table 2: Statistical data on material ultimate tensile strength

Material type	Product type	Source	No. of tests n	Thickness range (mm)	$f_{u,mean}$ (N/mm^2)	σ (N/mm^2)	COV	$f_{u,mean}/f_{u,min}$
Austenitic	C	[5]	2572	2.49-6.35	609	10.6	0.017	1.15
		Producer	-	-	639	23.0	0.034	1.18
	H	[7,8]	-	4.0	601	-	-	1.15
		Producer	-	-	613	14.3	0.023	1.17
	P	[6]	>3000	5.0-50	596	14.8	0.025	1.16
		[7,8]	-	15	580	-	-	1.13
		Producer	1368	-	600	17.4	0.029	1.15
		Producer	-	-	580	15.8	0.027	1.13
	Average				606			1.15
Duplex	C	[5]	239	2.49-6.35	812	12.1	0.015	1.23
		[9]	-	1.0	845	-	-	1.21
		Producer	5749	0.4-3.5	829	23.6	0.029	1.21
		Producer	-	-	806	28.1	0.036	1.18
		Producer	-	<6.4	752	21.0	0.028	1.07
	H	[9]	-	4.0	798	-	-	1.16
		Producer	-	-	775	28.7	0.038	1.16
		Producer	-	<10	718	7.0	0.010	1.06
	P	[6]	>300	5.05-50	763	13.7	0.018	1.19
		[9]	-	15	725	-	-	1.12
		Producer	-	-	742	18.8	0.025	1.16
	Average				775			1.16
Ferritic	C	Producer	-	-	493	17.6	0.036	1.16
		Producer	-	-	504	18.8	0.037	1.17
		Producer	-	>10	512	17.3	0.034	1.20
		Producer	438	1.25-2.0	500	12.1	0.024	1.16
	H	Producer	-	-	788	23.7	0.048	1.14
		Producer	-	-	512	30.4	0.059	1.14
	P	Producer	-	-	512	35.0	0.068	1.14
	Average				503			1.16

3.3 Statistical data on geometric properties

The dimensional variation of stainless steel elements is another source of variability in member resistance, and needs to be appropriately accounted for in the reliability analysis. In the absence of detailed records of dimensional variations from stainless steel section manufacturers, the required statistical information were obtained by studying the dimensional variation of test specimens from the collected database of structural performance data used in Section 4. Assuming that the test specimens are representative of sections used in practical applications, the magnitudes of all the key measured dimensions were compared against the corresponding nominal dimensions, enabling the determination of mean values and standard deviations for the ratios of the measured to nominal properties of different section types. Summaries of the obtained results for a total of 282 square and rectangular hollow sections (SHS and RHS), 74 circular hollow sections (CHS) and 62 I-sections are presented in Tables 3, 4 and 5, respectively.

The analysis indicated that, on average, sections tend to be marginally smaller than their nominal dimensions; however, the difference is considered insignificant and no correction for this discrepancy was included in the statistical reliability analysis, while due allowance for the obtained variability was made. Since the effect of the variability of the individual geometric parameters depends on the resistance function being considered, an overall coeffcient of variation $V_{geometry}$ parameter may be employed for different resistance functions. A method based on Equation (7) was used herein, where weighting factors associated with each geometric variable were evaluated, and used with the dimensional variation data presented in Tables 3, 4 and 5, to determine suitable $V_{geometry}$ parameters. Since the value of the weighting factors depend on the resistance function being considered, it is possible to have different $V_{geometry}$ values for a given section type used in different resistance functions. Values of $V_{geometry}$ were determined for SHS/RHS, CHS and I-sections for compression and bending loading cases. A summary of the results from this analysis is presented in Table 6. On a similar basis, Byfield and Nethercot [11] adopted a value of $V_{geometry}$ = 0.02 for carbon steel I-sections in compression and bending, while a larger value of $V_{geometry}$ = 0.05 was utilised for stainless steel in the development of the AISC stainless steel design guide [13]. Analysis of the results herein shows that $V_{geometry}$ = 0.05 is more appropriate for stainless steel sections; this value was adopted in all the reliability analyses carried out in this paper.

Table 3: Dimensional variation (i.e. ratios of mean to nominal values) of key dimensions of SHS and RHS

Dimension	Depth (h)	Breadth (b)	Thickness (t)
Mean	0.9999	1.0027	0.9755
Standard deviation	0.0205	0.0304	0.0362
Coefficient of variation	0.0205	0.0304	0.0362

Table 4: Dimensional variation (i.e. ratios of mean to nominal values) of key dimensions of CHS

Dimension	Outer diameter (D)	Thickness (t)
Mean	0.9853	0.9965
Standard deviation	0.0285	0.0138
Coefficient of variation	0.0289	0.0138

Table 5: Dimensional variation (i.e. ratios of mean to nominal values) of key dimensions of I-sections

Dimension	Depth (h)	Breadth (b)	Web Thickness (t)	Flange Thickness (t)
Mean	1.0141	0.9977	0.9991	0.9994
Standard deviation	0.0369	0.0132	0.0151	0.0182
Coefficient of variation	0.0364	0.0132	0.0151	0.0182

Table 6: Calculated values for the COV of geometric properties $V_{geometry}$ for stainless steel sections

Cross-section shape	Compression	Bending
SHS/RHS	0.0412	0.0486
CHS	0.0325	0.0606
I-section	0.0214	0.0495

4 DETERMINATION OF PARTIAL RESISTANCE FACTORS AND ASSESSMENT OF EN 1993-1-4

In this section, the reliability analysis procedures set out in Annex D of EN 1990 [2], as introduced in Section 2, along with the statistical data on material and geometric properties, presented in Section 3, have been applied to an extensive pool of structural performance data on stainless steels to assess the partial factors for the resistance functions provided in EN

1993-1-4 [1]. At the cross-section level, stub column and in-plane bending test results were used to assess the γ_{M0} partial resistance factor. At the member level, flexural buckling and lateral-torsional buckling test results were used to evaluate the γ_{M1} partial resistance factor. The classification of the cross-sections for the treatment of local buckling was based on the recent classification limits and effective width equations proposed by Gardner and Theofanous [14], which will replace the current guidelines in the forthcoming amendment to EN 1993-1-4 [1], which is due to be published in 2015.

4.1 Partial factor for cross-section resistance γ_{M0}

The compression resistance of a stainless steel cross-section $N_{c,Rd}$, as set out in EN 1993-1-4 [1], is given by Equation (15), where f_y is the material yield strength and A is the cross-sectional area, taken as the gross cross-sectional area for Class 1, 2 and 3 sections and the effective cross-sectional area A_{eff} for Class 4 sections. The design moment resistance of a cross-section subjected to uniaxial bending $M_{c,Rd}$ is given by Equation (16), where W is the appropriate section modulus, taken as the plastic section modulus W_{pl} for Class 1 and 2 sections, the elastic section modulus W_{el} for Class 3 sections and W_{eff} for Class 4 sections.

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{15}$$

$$M_{c,Rd} = \frac{Wf_y}{\gamma_{M0}} \tag{16}$$

Test data on stainless steel stub columns [12, 15–29] and beams [12, 17, 18, 26, 30–39] were collected and used to assess the partial factors γ_{M0} employed in Equations (15) and (16). Owing to the relatively simple form of these design resistance functions, Equation (8) was used to calculate the coeffcient of variation of the model V_{rt}, with the coeffcient of variation of the basic variables V_{Xi} taken as those presented in Section 3. The results of the statistical analysis for the two populations of data for cross-sections in compression and cross-sections in bending are reported in Tables 7 and 8, respectively. The $k_{d,n}$ parameter is the fractile factor, and is related to the number of tests in each data set. For SHS/RHS, $k_{d,n}$ was determined on the basis of the total pool of compression test data on this section type, including both stub columns and long columns. A similar approach was taken for I-sections and CHS, while the test data on angle, channel and lipped channel sections were combined to determine a common $k_{d,n}$ value for these sections. The resulting values of $k_{d,n}$ are reported in Table 7.

The required values of γ_{M0} for cross-section compression resistance derived from the statistical analyses are reported in Table 7. For SHS/RHS and I-sections, the current γ_{M0} value of 1.1 is found to be sufficient for all stainless steel grades considered. Test data on stainless steel open sections such as channles, lipped channels and angles, are relatively limited, and the data used in this study were acquired from a single source [24], based on which it is indicated that a γ_{M0} value higher than 1.1 may be required. However, it is recommended that the current γ_{M0} value of 1.1 should be maintained for these section types in the absence of a comprehensive set of structural performance test or FE data. Analysis of the CHS data suggests that while γ_{M0} = 1.1 is conservative for the case of duplex and ferritic grades, it needs to be increased for the case of austenitic stainless steels. A high γ_{M0} value of 1.32 for the austenitic grade is mainly as a result of a combination of low b and high V_δ values for this material. Figure 2 shows the results of all CHS test data, including long columns, where the reduction factor $\chi = N_{test}/Af_y$ is plotted against the member slenderness $\bar{\lambda}$. It shows that the current plateau length of $\bar{\lambda}_0 = 0.4$ as adopted in EN 1993-1-4 [1], below which member buckling checks are not required, is rather optimistic for CHS members. This elongated plateau length influences the results of the statistical analysis on cross-section compression resistance, and contributes to a high required value of γ_{M0} for the austenitic material, which features test data towards the end of the plateau. The member buckling curves given in of EN 1993-1-4 [1] for the design of stainless steel compression members are known to require reconsideration [40]; this is the subject of ongoing research and will be discussed further in subsequent sections.

The statistical analysis results presented in Table 8 suggest that the EN 1993-1-4 [1] design resistance equation for cross-section bending capacity is consistently conservative for austenitic, duplex and ferritic stainless steels, and considering the γ_{M0} value for all cross-section types and grades included for this loading type, it is proposed that the current value of 1.1 is maintained.

Table 7: Summary of statistical analysis results for cross-section compression resistance.

Section type	Material	No. of tests n	b	Over-strength	$k_{d,n}$	V_δ	V_{fy}	$V_{geometry}$	γ_{M0}
SHS/RHS	Austenitic	71	1.245	1.30	3.14	0.156	0.060	0.05	1.08
I-section	Austenitic	20	1.067	1.30	3.30	0.099	0.060	0.05	1.09
Angle	Austenitic	12	1.122	1.30	3.40	0.110	0.060	0.05	1.07
Channel	Austenitic	11	1.099	1.30	3.40	0.125	0.060	0.05	1.15
Lipped channel	Austenitic	12	0.974	1.30	3.40	0.088	0.060	0.05	1.16
CHS	Austenitic	19	0.968	1.30	3.23	0.135	0.060	0.05	1.32
SHS/RHS	Duplex	24	1.143	1.10	3.14	0.083	0.030	0.05	1.10
I-section	Duplex	5	1.202	1.10	3.30	0.032	0.030	0.05	1.06
CHS	Duplex	7	1.295	1.10	3.23	0.032	0.030	0.05	0.86
SHS/RHS	Ferritic	9	1.073	1.20	3.14	0.054	0.045	0.05	1.02
I-section	Ferritic	7	1.099	1.20	3.30	0.044	0.045	0.05	0.98
CHS	Ferritic	4	1.182	1.20	3.23	0.036	0.045	0.05	0.90

Table 8: Summary of statistical analysis results for cross-section bending resistance.

Section type	Material	No. of tests n	b	Over-strength	$k_{d,n}$	V_δ	V_{fy}	$V_{geometry}$	γ_{M0}
SHS/RHS	Austenitic	45	1.296	1.30	3.25	0.120	0.060	0.05	0.95
I-section	Austenitic	5	1.136	1.30	4.08	0.056	0.060	0.05	0.94
CHS	Austenitic	8	1.272	1.30	4.33	0.122	0.060	0.05	1.08
SHS/RHS	Duplex	12	1.219	1.10	3.25	0.095	0.030	0.05	1.07
I-section	Duplex	8	1.342	1.10	4.08	0.089	0.030	0.05	1.02
CHS	Duplex	3	1.319	1.10	4.33	0.011	0.030	0.05	0.83
SHS/RHS	Ferritic	8	1.116	1.20	3.25	0.057	0.045	0.05	0.99

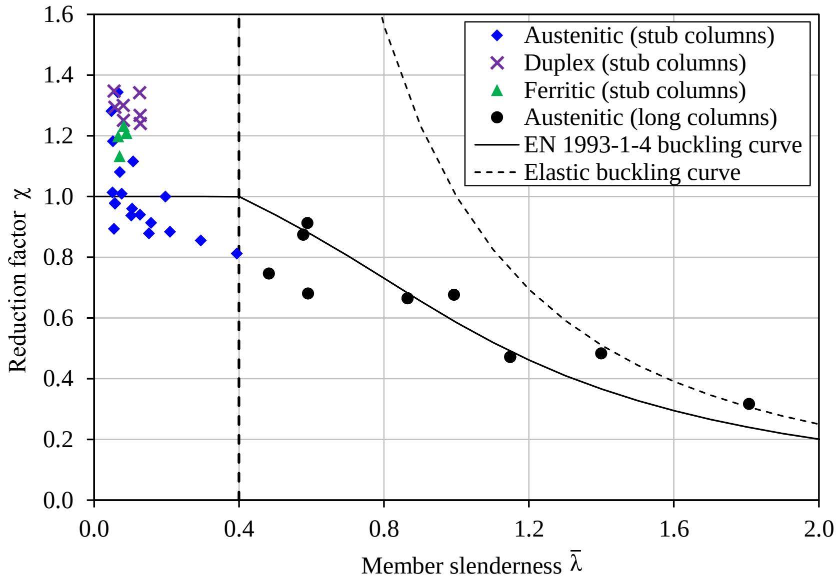

Figure 2: Comparison of CHS compression test data with EN 1993-1-4 buckling curve

4.2 Partial factor for member resistance γ_{M1}

4.2.1 Flexural buckling resistance

The flexural buckling resistance of a stainless steel compression member $N_{b,Rd}$, as set out in EN 1993-1-4 [2], is given by Equation (17), where f_y is the material yield strength, A is the cross-sectional area (taken as the gross cross-sectional area for Class 1, 2 and 3 sections and effective cross-sectional area A_{eff} for Class 4 sections), γ_{M1} is the partial resistance factor for member resistance and χ is the flexural buckling reduction factor, determined from Equation (18).

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \text{ for } \bar{\lambda} > \bar{\lambda}_0 \tag{17}$$

in which the flexural buckling reduction factor χ is given by:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0 \text{ with } \phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - \bar{\lambda}_0\right) + \bar{\lambda}^2\right] \text{and } \bar{\lambda} = \sqrt{Af_y / N_{cr}} \tag{18}$$

where N_{cr} is the elastic critical buckling load, α is the imperfection factor and $\bar{\lambda}_0$ is the non-dimensional limiting slenderness (i.e. the plateau length). For cold-formed open sections and hollow sections, $\bar{\lambda}_0 = 0.4$ and α = 0.49, for welded open sections (buckling about the major axis) $\bar{\lambda}_0 = 0.2$ and α = 0.49 and for welded open sections (buckling about the minor axis) $\bar{\lambda}_0 = 0.2$ and α = 0.76.

In order to separate the dependency of the buckling reduction factor χ on the other basic variables in the design model, f_y and A, given in Equation (17), the resistance function may be expressed as given in Equation (19) where, k is the model constant, independent of A and f_y, and c and d are the model parameters specific to each test specimen and vary with column slenderness $\bar{\lambda}$.

$$N_{b,Rd} = k f_y^{\,c} A^d \tag{19}$$

The approach to determine the parameters c and d for each specific test specimen are outlined herein. Considering two columns with the same cross-sectional area A and different yield strength values $f_{y,1}$ and $f_{y,2}$, using Equation (19) the ratio of their capacities becomes:

$$\frac{N_{b,Rd,2}}{N_{b,Rd,1}} = \frac{kf_{y,2}{}^{c}A^{d}}{kf_{y,1}{}^{c}A^{d}} = \left(\frac{f_{y,2}}{f_{y,1}}\right)^{c} \tag{20}$$

Hence, c may be determined as:

$$c = \frac{\ln\left(N_{b,Rd,2}/N_{b,Rd,1}\right)}{\ln\left(f_{y,2}/f_{y,1}\right)} \tag{21}$$

The power d may subsequently be determined from Equation (22) by considering two columns of differing cross-sectional area A_1 and A_2, assuming that the section second moment of area I is approximately proportional to A_2, giving $N_{cr,1}/N_{cr,2} \approx (A_1/A_2)^{2}$.

$$d = \frac{\ln\left(N_{b,Rd,2}/N_{b,Rd,1}\right) - c\ln\left(f_{y,2}/f_{y,1}\right)}{\ln\left(A_2/A_1\right)} \tag{22}$$

The model parameters c and d were evaluated for each test data using Equations (21) and (22), respectively by considering a small increase in the variable being changed, i.e. taking $f_{y,2} = 1.001f_{y,1}$ and $A_2 = 1.001A_1$. The relationship between the two powers c and d and the non-dimensional slenderness $\bar{\lambda}$ has been plotted in Figure 3. The values of the c and d parameters were calculated based on a plateau length of $\bar{\lambda}_0 = 0.4$ and imperfection factor of $\alpha = 0.49$, which correspond to the buckling curve specified in EN 1993-1-4 [1] for cold-formed open sections and hollow sections (welded and seamless). At low slenderness values, $\bar{\lambda}_0 \leq 0.4$, column capacity is limited by the cross-section resistance which is controlled by the material yield strength f_y and the cross-sectional area A, as presented in Equation (15), and therefore c = d = 1. Note that in this instance, Equation (7) simplifies to Equation (8). At higher slenderness values, $\bar{\lambda}_0 > 0.4$, the column buckling load $N_{b,Rd}$ approaches the elastic buckling load N_{cr}, which is independent of f_y, but dependent on the section geometry; hence the parameter c approaches zero and $N_{b,Rd}$ will only depend on the geometric properties and may be expressed as $N_{b,Rd} = kf_y^{0}A^{d}$. It is shown in Figure 3 that d approaches a value of 2.0 with increasing member slenderness $\bar{\lambda}$, which coincides with the elastic critical buckling load N_{cr} considering that the second moment of area I was taken as approximately proportional to A^2. In addition, owing to the complex form of the flexural buckling resistance formulation provided in EN 1993 1-4 [1], the V_{rt} parameter, used to allow for the variability of the material and geometric basic variables, was determined from Equation (7). This allows for the varying degree of the influence of the basic variables f_y and A at different values of member slenderness to be taken into account. Flexural buckling test data collected from [16, 17, 19–21, 27, 28, 301, 32, 41, 42], were analysed following the above described modified approach, and values of the partial factor γ_{M1} for each test specimen were determined. From the least squares regression of the individual values obtained, an overall γ_{M1} value was subsequently determined for each stainless steel type considered - see Equation (23).

$$\gamma_{M1} = \frac{\sum_{i=1}^{n} r_{n,i}^{2}}{\sum_{i=1}^{n} r_{n,i} r_{d,i}} \tag{23}$$

where, $r_{n,i}$ is the nominal resistance, based on the EN 1993-1-4 [1] flexural buckling design equation and a nominal f_y value, and $r_{d,i}$ is the design resistance from Equation (9), both evaluated for each test specimen.

The nominal yield strength may be taken as the minimum specified yield strength, provided in EN 10088-4 [10]. However, this approach was considered unsatisfactory in the analyses carried out in this paper, as the minimum specified strength may not be representative of the nominal strength of the material in the test programme considered, resulting in overly conservative partial factors. Therefore, the nominal strength in this study was taken as the mean strength, from measured test data, reduced by the relevant over-strength factor, e.g. $f_{y,nom} = f_{y,mean}$/(over-strength factor). A summary of the key results of the reliability analysis is presented in Table 9. Values of the attained partial factors γ_{M1} that are greater than 1.1 indicate that the current EN 1993-1-4 [1] column buckling curve fails to meet the Eurocode reliability requirements. For SHS/RHS columns, the results indicate that a slightly lower buckling curve may be required; a similar conclusion was reached for the case of ferritic stainless steels in [17], where alternative lower buckling curves were proposed. Considering that the scatter of the test data is not particularly high (see Figure 4), and also the relatively large number of test results in this category, it is unlikely that this result would change if further testing was carried out. Therefore, it is recommended that lower buckling curves for SHS/RHS members are developed. The results of Table 9 also suggest that the current provisions for austenitic circular hollow sections (CHS) are unsafe. The reason for this result can be seen in Figure 2, where between slenderness values of 0.2 and 0.6, several data points are substantially below the buckling curve.

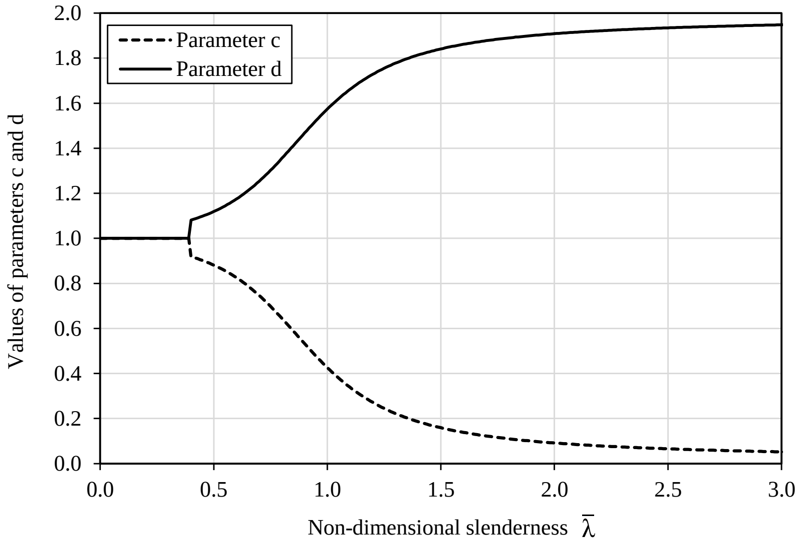

Figure 3: The powers a or b versus non-dimensional slenderness $\bar{\lambda}$.

Table 9: Summary of statistical analysis results for flexural buckling resistance.

Section type	Material	No. of tests n	b	Over-strength	$k_{d,n}$	V_δ	V_{fy}	$V_{geometry}$	γ_{M0}
SHS/RHS	Austenitic	67	1.070	1.30	3.14	0.094	0.060	0.05	1.16
I-section	Austenitic	14	1.008	1.30	3.30	0.070	0.060	0.05	1.13
CHS	Austenitic	12	0.985	1.30	3.23	0.168	0.060	0.05	1.57
SHS/RHS	Duplex	25	1.062	1.10	3.14	0.075	0.030	0.05	1.22
I-section	Duplex	3	1.026	1.10	3.30	0.009	0.030	0.05	1.13
SHS/RHS	Ferritic	14	0.984	1.20	3.14	0.070	0.045	0.05	1.24

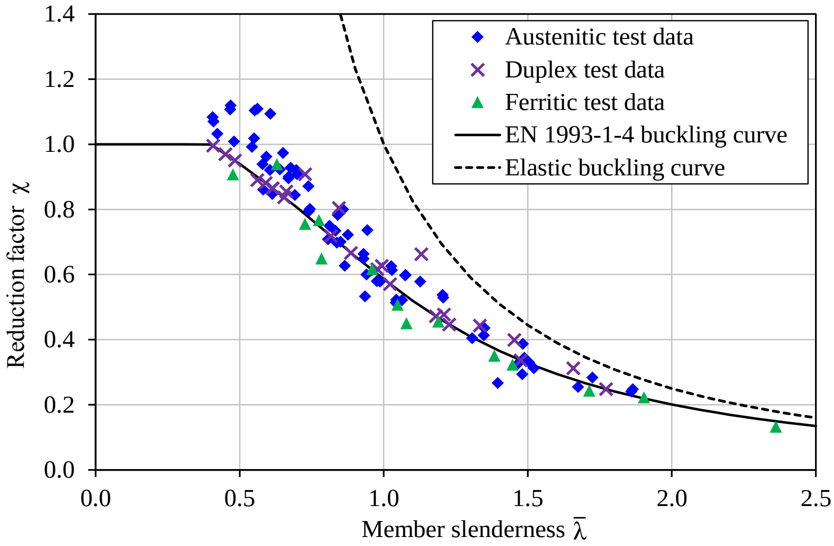

Figure 4: SHS/RHS column buckling test data and EN 1993-1-4 buckling curve.

4.2.2 Lateral-torsional buckling

EN 1993-1-4 [1] defines the lateral-torsional buckling resistance of laterally unrestrained beams through Equation (24), where W_y is the major axis section modulus, taken as the plastic section modulus $W_{pl,y}$ for Class 1 and 2 sections, the elastic section modulus $W_{el,y}$ for Class 3 sections and effective section modulus $W_{eff,y}$ for Class 4 sections.

$$M_{b,Rd} = \frac{\chi_{LT} W_y f_y}{\gamma_{M1}} \tag{24}$$

in which the lateral-torsional buckling reduction factor χ_{LT} is given by:

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \overline{\lambda}_{LT}^2}} \leq 1.0 \text{ with } \phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\overline{\lambda}_{LT} - 0.4\right) + \overline{\lambda}_{LT}^2\right] \text{and } \overline{\lambda}_{LT} = \sqrt{W_Y f_y / M_{cr}} \tag{25}$$

where M_{cr} is the elastic critical buckling moment, α_{LT} is the imperfection factor, taken as 0.34 for cold-formed open sections and hollow sections (welded and seamless) and 0.76 for welded open sections.

To separate the dependency of the design equation on the basic variables, the resistance function was expressed as in Equation (26), where e is determined from Equation (27), following a similar procedure as described for flexural buckling, and f was taken as unity. This enabled the determination of V_{rt} for lateral-torsional buckling.

$$M_{b,Rd} = k f_y^{\,e} W_y^{\,f} \tag{26}$$

$$e = \frac{\ln\left(M_{b,Rd,2} / M_{b,Rd,1}\right)}{\ln\left(f_{y,2} / f_{y,1}\right)} \tag{27}$$

The above described method was applied to lateral-torsional buckling test data obtained from [12, 43, 44], and a summary of the statistical analysis results is provided in Table 10. The calculated values of γ_{M1} suggest that a higher partial factor that the current value 1.1 or a lower buckling curve is necessary for lateral-torsional buckling. The results from these tests show relatively high scatter (see Figure 5), perhaps due to the manner in which the tests were conducted, but very few points lie below the design curve and those that do are only marginally below. Hence, the current buckling curve is considered to be satisfactory.

Table 10: Summary of statistical analysis results for lateral-torsional buckling resistance.

Section type	Material	No. of tests n	b	Over-strength	$k_{d,n}$	V_δ	V_{fy}	$V_{geometry}$	γ_{M0}
I-section	Austenitic	14	1.066	1.30	3.36	0.112	0.060	0.05	1.19
I-section	Ferritic	16	1.368	1.20	3.36	0.152	0.045	0.05	1.13

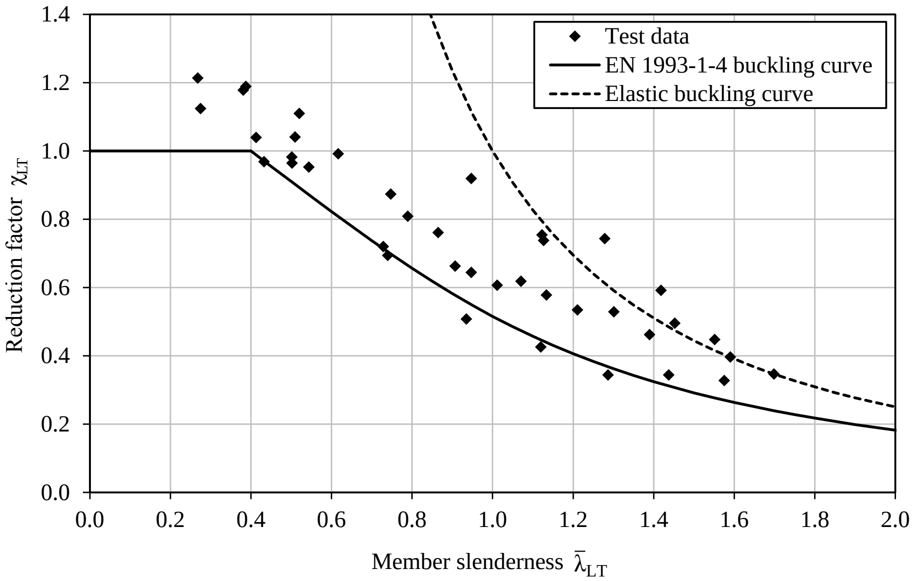

Figure 5: Lateral-torsional buckling test data with EN 1993-1-1 buckling curve.

5 CONCLUSIONS

A reliability assessment of the EN 1993-1-4 structural stainless steel design provisions has been carried out in this study, and the obtained results have been presented and discussed. Statistical data on material properties suitable for use in reliability analyses were derived from industrial data. For yield strength, representative over-strength values and COVs of 1.3 and 0.060 for austenitic, 1.1 and 0.030 for duplex and 1.2 and 0.045 for ferritic stainless steels were established, while for the ultimate tensile strength, an over-strength value of 1.1 for all stainless steel grades and COVs of 0.035 for the austenitic and duplex grades and 0.05 for the ferritic grade were proposed. Based on the database of sections considered in this study, a COV value of 0.05 was adopted to represent the variability of the geometric properties. Analysis of cross-section compression and in-plane bending test results showed that the current γ_{M0} value of 1.1 given in EN 1993-1-4 may be maintained for the section types considered, excluding CHS elements in compression, where revised design provisions are needed and a shorter plateau length is recommended. Column flexural buckling design rules were also assessed, and it was found that the current γ_{M1} value of 1.1 is generally satisfactory, but some buckling curves, particularly for CHS compression members, should be revisited. For cases of lateral–torsional buckling, it was recommended that the current γ_{M1} = 1.1 is maintained, but a reassessment of this value needs to be carried out upon generation of a more comprehensive pool of experimental data.

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