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DEVELOPMENT OF A DESIGN METHODOLOGY FOR SLENDER CARBON-REINFORCED CONCRETE COLUMNS IN AXIAL COMPRESSION BASED ON EC3

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Abstract. Advancements in concrete construction, such as carbon-reinforced and ultra-high performance concretes, enable the creation of slender, high-capacity structures, enhancing resource efficiency and reducing CO_2 emissions. Despite the clear advantages of such innovative material composites, challenging load-bearing and deformation behavior emerges in slender carbon-reinforced concrete components, indicating potential stability issues. To address this concern, current research is dedicated to experimental and analytical investigations of the structural behavior and failure of slender components in compression made of carbon-reinforced concrete, aiming to enhance our understanding of stability-related aspects.

This article focuses on a specific approach to derive a practical design procedure for carbon-reinforced concrete columns in compression based on results from the literature. Following the hypothesis that design concepts of steel construction are transferrable to slender CRC structures, a suggestion for a verification format for buckling is suggested. The proposed methodology follows the proven design concept of steel construction, where the (plastic) resistance of the cross-section for uniform compression is reduced by a reduction factor χ. The advantage of this design principle is that the verification of the cross-sectional capacity is expanded by only one factor to consider buckling appropriately. To enable such an approach for slender carbon-reinforced concrete columns, buckling curves are derived, related slenderness limits and adjustment factors are determined. It is suggested to relate the capacity of the column to a slenderness-related strength at $\lambda_{lim} = 25$ instead of the uniaxial concrete compressive strength ($\lambda << 25$). In this way, a general representation of buckling curves is achieved, which can be applied to a broader range of columns. The overarching goal is to contribute to the development of safe and efficient construction methods with carbon-reinforced concrete, promoting the application of these innovative material composites and facilitating their integration into construction practices.

1 INTRODUCTION

Reducing global CO_2 emissions and conserving natural resources are critical to mitigating climate change and ensuring environmental sustainability. This need arises from the increasing pressure on the environment caused by anthropogenic influences, in particular the emission of greenhouse gases such as CO_2. The construction sector is a major contributor to this burden, as it is responsible for a significant proportion of global CO_2 emissions. Approximately 40% of global CO_2 emissions [1] and as much as 50% of resource consumption [2] can be attributed to the construction sector.

A significant portion of a building's CO_2 emissions are generated during its construction in the form of so-called embodied carbon. These are mainly the result of the use of building materials such as concrete or cement, the production of which causes high CO2 emissions. Reducing material consumption is therefore a key starting point for reducing environmental impact. Material efficiency not only means reducing embodied carbon, but also conserving natural resources.

The use of carbon-reinforced concrete (CRC) or textile-reinforced concrete (TRC) offers a promising solution for saving large quantities of concrete. CRC can be used for strengthening and retrofitting of existing structures [3–7] as well as for newly built constructions [8–13]. Beside reinforcement for concrete structures, carbon or carbon fiber reinforced polymer (CFRP) can be used for strengthening of steel constructions, too [14, 15]. By using endless carbon fibers or carbon textiles as reinforcing material, slender and filigree concrete members can be realized. As a result, the stability behavior of the structures made of CRC or TRC becomes more important compared to the ones of steel-reinforced concrete (RC). This phenomenon that stability behavior is more important than material behavior is already widespread in steel construction, where components are generally very slender, and the stability behavior is a central aspect in the verification concept according to EC3.

The transfer of methods from steel construction to carbon-reinforced concrete construction therefore opens up interesting perspectives. The question is how carbon-reinforced concrete construction can learn from steel construction, especially with regard to the design and verification of slender compression members. The integration of knowledge and methods from steel construction into the verification concept for CRC structures could make an important contribution to the optimization of the construction method and the further reduction of the environmental impact.

2 CONSIDERATION OF BUCKLING IN EC2

2.1 Consideration of stability for steel-reinforced concrete

Pure buckling failure is not considered a critical limit state in reinforced concrete due to the imperfections to be considered. Instead, the proof of stability for components at risk of stability under normal force is calculated according to the second order approaches. The calculation of internal forces for slender structures or components that are mainly subjected to longitudinal pressure and whose deformations significantly influence their resistance is governed by Eurocode 2 (EC2), Section 5.8. Accordingly, the effects according to second-order theory must be taken into account if they amount to more than 10% of the effects according to the first-order theory. The proof on the deformed system, which provides information about the increase in bending moment, is elaborate and not very often required for RC structures. Alternatively, a simplified criterion for individual compression members can be used to assess the risk of buckling if the slenderness of a member under compression does not exceed the limit value λ_{lim} (see EC2, para. 5.8.3.1.(1)). In the associated National Annex, this is defined as $\lambda_{lim} = 25$. Since the influence of the deformations on the load-bearing capacity of the component also decreases with decreasing utilisation of the cross-section, the limit value for related normal forces $n < 0.41$ may be reduced. If this simplification is not possible, i.e. if the second-order effects need to be considered, EC2 provides three methods for the calculation of compression members. In addition to the non-linear calculation on the overall system, these are two simplified methods based on the use of nominal curvatures or nominal stiffnesses. In these so-called equivalent member methods, the compression members are analysed in a simplified manner as individual cantilever columns (Euler case 1) detached from the overall structure for loaded member internal forces. In Germany, due to the partial inefficiency of the nominal stiffness method, the nominal curvature method has tended to prevail. This method is also known as the model support method from the German standard DIN 1045-1. This approximate calculation, which converts the complicated and time-consuming iterative calculation process according to second-order theory into a simple cross-section design, is sufficiently accurate for the support systems commonly used in building construction and, as an easy-to-use method, also enables calculation by hand.

2.2 Consideration of stability for carbon-reinforced concrete

Although numerous design approaches have already been developed in the course of many years of research on CRC, there has been a lack of a standard with normative rules for this material for a long time. Since 2018, a consortium of over 30 partners from industry, research and authorities, led by the German Committee for Reinforced Concrete (Deutscher Ausschuss fuer Stahlbeton, DAfStb), has been working on the creation of a guideline for "Concrete components with non-metallic reinforcement" [16]. The white print of this guideline for nonmetallic-reinforced concrete was published in January 2024. Part 1, which regulates the structural design and implementation, has the same structure as EC2 German Version EN 1992-1-1 [17]. Thus, the corresponding sections of this standard and its national annex [18] apply, unless otherwise specified in the guideline. According to the DAfStb guideline for non-metallic reinforced concrete, the rule applies to non-metallic reinforced concrete components under normal force that effects according to second-order theory do not have to be taken into account if they are less than 10% of the effects according to first-order theory. This regulation is analogous to the regulation for steel-reinforced concrete. However, for non-metallic reinforced concrete, a simplified verification for individual compression members based on the limit slenderness criterion λ_{lim} may not be used. The regulations from EC2 initially apply for the application of imperfections. Currently, only the general method based on a non-linear determination of internal forces according to second-order theory is permissible. In contrast to steel-reinforced concrete, the simplified method with nominal curvature (model column method) is not yet permitted for non-metallic reinforced concrete according to the DAfStb guideline. The approach for steel-reinforced concrete is not directly transferable to carbon-reinforced concrete due to the different material behavior of non-metallic reinforcement. In addition, there are still specifications missing, e.g. with regard to the maximum curvature to be applied or a reasonable limitation of the reinforcement strain. In addition, it still needs to be investigated whether larger imperfections should be assumed for carbon-reinforced concrete than for steel-reinforced concrete components or whether they may even be reduced due to high precision during production [19].

3 VERIFICATION FORMAT FOR BUCKLING IN EC3

The verification format in EC3 [120] for buckling is based on an extensive experimental test program conducted by the European Convention for Constructional Steelwork over a period of 15 years. The results of this research yielded curves for the specific buckling behavior of various cross-section types, which were used to create a representation of the limit load as a function of slenderness. These curves were made calculable using an analytical approach by Maquoi and Rondal [21]. They described the existing buckling tests with the model of a column of length l and loaded with a normal force N_E using the second-order theory with an imperfection figure affine to the first eigenmode and the amplitude η_{init}. The solution was derived from the differential equation of the bending line for the model shown in Fig. 1(a).

$$\text{Differential equation: } EI_z\eta'''' + N_E\eta'' = 0 \tag{1}$$

$$\text{Solution: } N_{cr} = EI_z\frac{\pi^2}{l^2};\ \eta_{cr} = \sin\frac{\pi x}{l} \tag{2}$$

$$\text{Imperfection: } \eta_{init} = e_0\frac{N_{cr}}{EI_z\eta''_{cr,max}}\eta_{cr} = e_0\sin\frac{\pi x}{l} \tag{3}$$

$$\text{Secondary bending moment resulting from second-order theory: } M_E^{II} = N_E e_0\frac{1}{1-N_E/N_{cr}} \tag{4}$$

Here, N_{cr} is the elastic critical buckling load, η_{cr} is the shape of the eigenform and $EI_z\eta''_{cr,max}$ is the bending moment due to η_{cr} at the critical cross-section. Maquoi and Rondal choose the following approach for the amplitude of the geometric imperfection e_0:

$$e_0 = \frac{M_R}{N_R}(\bar{\lambda} - \bar{\lambda}_{limit})\alpha = \frac{M_R}{N_R}(\bar{\lambda} - 0{,}2)\alpha \tag{5}$$

Here, M_R/N_R is a factor from the cross-sectional shape, $(\bar{\lambda} - 0{,}2)$ is a factor from the slenderness, and α is the imperfection factor. The imperfection factor considers all other parameters that are not included in the simplified model, such as structural imperfections, as well as model inaccuracies. The objective is to adjust the results to the values of the statistical distribution of the test results [22]. The buckling coefficients were determined in such a way that a non-strict order relation could be established with the test results. Consequently, the approach of e_0 is also justified in terms of reliability and is suitable for integration into the semi-probabilistic design concept of the Eurocodes. For non-dimensional slenderness of $\bar{\lambda} \leq 0{,}2$, buckling verification may be omitted, respectively, a cross-sectional failure of the compression member can be assumed [20].

The verification form for compression members in EC3 [20] is based on a linear interaction of the normal force and moment cross-sectional resistance, see Fig. 1(b). The moment is logically derived from the additional moment resulting from the second-order theory, as outlined in Eq. (4).

$$\frac{N_E}{N_R} + \frac{M_E}{M_R} = \frac{N_E}{N_R} + \frac{N_E e_0}{M_R}\frac{1}{1 - N_E/N_{cr}} \leq 1 \tag{6}$$

From this approach, and taking into account the amplitude of the geometric imperfection e_0 from Eq. (5), the reduction factor for the buckling curve χ is derived.

$$\chi = \chi(\alpha, \bar{\lambda}) = \frac{1}{\varphi + \sqrt{\varphi^2 - \bar{\lambda}^2}} \tag{7}$$

$$\varphi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right] \tag{8}$$

The outcome of this methodology is the formula for the buckling curves with the corresponding imperfection coefficient α, see Fig. 1(c), and the straightforward verification form, in which the cross-sectional load-bearing capacity of the compression member is reduced by χ. The various buckling curves (a0 to d) illustrate the diverse cross-sectional shapes and buckling about the distinct cross-sectional axes.

$$N_{Ed} \leq N_{Rd} = \frac{\chi \cdot N_{Rk}}{\gamma_M} \tag{9}$$

A significant benefit of the presented methodology is the dimensionless representation of the buckling curves through the introduction of a non-dimensional slenderness $\bar{\lambda}$.

$$\bar{\lambda} = \sqrt{\frac{N_R}{N_{cr}}} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \tag{10}$$

Here, $\alpha_{ult,k}$ is minimum load amplifier of the load N_E to reach the resistance N_R of the most critical cross section of the structural component considering its in plane behavior without taking lateral or lateral torsional buckling into account however accounting for all effects due to in plane geometrical deformation and imperfections, global and local, were relevant and α_{cr} is minimum force amplifier to reach the elastic critical buckling load N_{cr}.

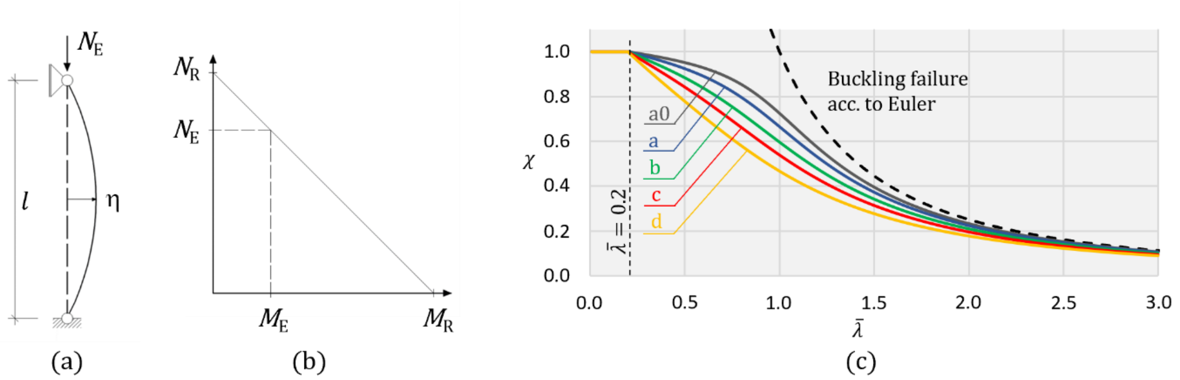

Figure 1: (a) buckling model for compression members; (b) linear verification model; (c) buckling curves acc. to EC3

4 EXPERIMENTAL STUDIES ON SLENDER COLUMNS

4.1 Investigations of the influence of the cross-sectional shape of CRC columns

In [23] buckling tests on slender CRC columns with different cross-sectional shapes are presented. The test specimens were produced using a fine-grained concrete, the material properties of which were determined on cylinders with a diameter of 100 mm and a height of

200 mm. The compressive strength of the concrete was found to be 53.5 MPa and the Young's modulus was determined to be 27.6 GPa. Carbon fibers (CF) with a tensile strength of 1140 MPa and a Young's modulus of 189 GPa were utilized as the reinforcement material.

The stability behavior of CRC columns is investigated in compression tests in [23] for I-shaped cross-sections 100/50/12/12 ($h/b/t_f/t_w$) with different lengths, ranging from 40 cm to 120 cm. The reinforcement ratio for the I-shaped columns is 1.94 %. Moreover, rectangular cross-sections with dimensions of 10x70 mm² and a reinforcement ratio of 2.37 % were subjected to compression tests, with lengths ranging from 60 cm to 120 cm. Dos Santos presents the results of these tests in a dimensionless form, as illustrated in Fig. 1(c).

4.2 Investigations on slender columns made of UHPC

Schmidt investigates the buckling behavior of slender columns made of UHPC in [24, 25]. The material properties were determined in deviation from DIN EN 12390 and DIN EN 196-1 due to the limited mixing capacity. For this reason, the compressive strength f_{cm} of 142 MPa is given as the equivalent cylindrical compressive strength. Experimental tests were performed on compact compression members with a rectangular cross-section of 94x94 mm² and buckling lengths of 75 cm (λ = 20) and 95 cm (λ = 28). Modulus of elasticity and compressive strength are determined for these compact specimens.

The buckling behavior was investigated in the laboratory for different rectangular cross-sections (94x95 mm², 72x74 mm², 70x102 mm², and 62x80 mm²) with lengths ranging from 363 cm to 380 cm. Schmidt presents the results of the compression tests on slender UHPC columns in an ultimate stress-slenderness ratio diagram.

4.3 Investigations of the influence of the load eccentricity of CRC columns

In [26], the influence of load eccentricity on the buckling behavior of slender CRC columns is experimentally investigated and discussed. A matrix of fine-grained concrete and CF reinforcement (f_u = 4048 MPa, E = 245 GPa) with a reinforcement ratio of 0.3% is used. The compressive strength of the concrete is determined experimentally on rectangular specimens 160x40x40 mm³ with f_{cm} = 111.5 MPa and the modulus of elasticity on cylinders Ø150 mm/300 mm with 43.4 GPa. Giese et al. [26] perform compression tests on compact specimens 200x100x26 mm³ with a slenderness ratio of 26.6 and determines the ultimate stress at 105 MPa and the modulus of elasticity at 43.6 GPa.

Furthermore, buckling tests on rectangular CRC columns with a cross-sectional area of 105x26 mm² are presented for three different buckling lengths of 50 cm, 68 cm, and 98 cm and three eccentricities of 0 mm, 2 mm, and 4 mm. The results are presented in the form of an ultimate stress-to-slenderness ratio diagram that distinguishes between buckling failure and material failure due to compression. An analytical failure envelope for carbon-reinforced concrete under combined bending and longitudinal force is derived in [27], using an adapted, linear-elastic material model for the non-metallic reinforcement.

5 APPLICATION OF THE EC3-BASED DESIGN PRINCIPLE FOR SLENDER CRC COLUMNS

The aim of this article is to investigate how the test results of slender CRC columns can be classified according to the EC3 principle and which conditions have to be fulfilled. The background is the transition from a material-driven failure to a stability failure of compressed, slender CRC components when using UHCP.

The basic approach in steel construction is to relate the slenderness ratio λ to the material properties λ_1, which is expressed by the related non-dimensional slenderness $\bar{\lambda}$.

$$\bar{\lambda} = \frac{L_{cr}}{i \cdot \lambda_1} \tag{11}$$

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} \tag{12}$$

To transfer this to the load-bearing behavior of slender CRC columns, it is useful to refer to the material behavior or specimen behavior of compact compression members. In this way, material related and stability failures can be better separated in the model. According to EC2, second-order effects must be considered if they are 10% larger than those of the first-order theory. For single compression components, this condition is described in the form of a slenderness limit value for easier handling. Taking into account the German National Annex, a limit slenderness of λ_{limit} = 25 applies. This means the following expression for the related slenderness, taking into account the mechanical parameters determined by tests.

$$\bar{\lambda} = 25 \frac{1}{\pi \sqrt{\frac{E(\lambda_{limit})}{f_{cm}(\lambda_{limit})}}} \tag{13}$$

The test boundary conditions, the main input variables and the results for the corresponding limit slenderness are summarized in Table 1 according to the tests presented in [23], in [24, 25] and in [26].

Table 1: Summary of test boundary conditions, input variables, and related limit slendernss

	material parameters derived from tests on compact compression members		tests on slender columns		
	$E(\lambda \approx 25)$ [GPa]	$f_{cm}(\lambda \approx 25)$ [MPa]	e [mm]	l [cm]	$\bar{\lambda}_{limit}$ [-]
dos Santos [23]	N/A	N/A	N/A	40-120	N/A
Schmidt [24, 25]	45.4	96.2	N/A	363-380	0.37
Giese et al. [26]	43.7	105	0-4	50-98	0.39

Instead of the upper slenderness limit of 0.2 given in EC3, this approach gives an average value of 0.38 for the slender CRC columns studied. This limit value thus determines the kink of the buckling curves based on Maquoi and Rondal [21]. The imperfection coefficient α introduced in Eq. (5) is a coefficient describing the shape of the curve to be determined by comparison with the results of the compression tests. The buckling curves shown in Fig. 2 can be derived for the buckling tests presented in [23], in [24, 25] and in [26].

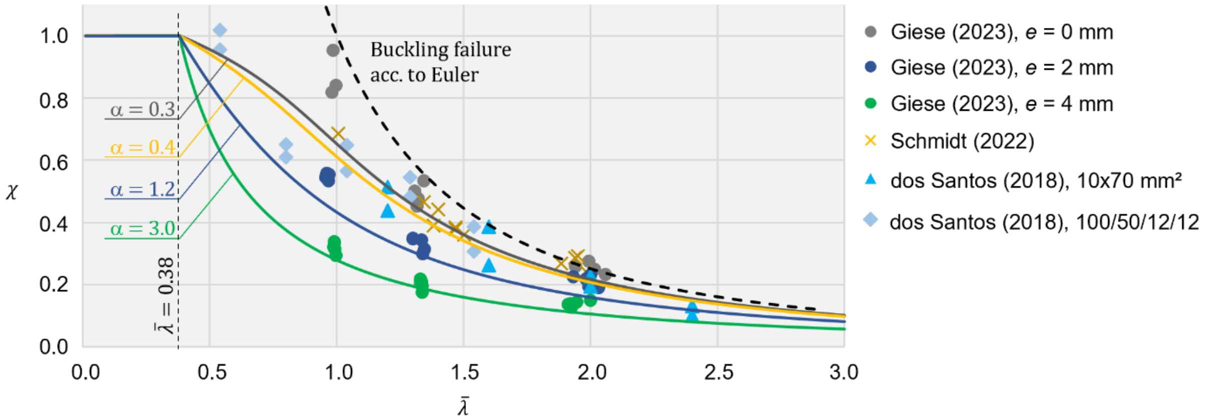

Figure 2: Buckling curves and test results from [23], [24, 25] and [26]

In steel construction, the imperfection coefficients α can be assigned to different profile types, e.g., welded and rolled profiles, and to different section shapes, e.g., I-, T-, and U-sections, angles, hollow sections, and so on. This is associated with different and profile-specific structural imperfections, which are mainly due to the production of the profiles. There is still no knowledge about the structural imperfections of CRC columns, so such a classification cannot yet be made on the basis of the test data shown. The simplified approach for the geometric imperfection with an eccentricity of $e_i = l/400$ according to EC2 can provide a first approach for the classification of buckling curves. This value is defined to cover imperfections related to normal execution deviations. In order to demonstrate the impact of the buckling curve definition on the design methodology, this work also considers the double value for the imperfection with $l/200$. The fitting of the buckling curves to these imperfections is shown in Fig. 3. From a comparison with the experimental results (see Chapter 3), a deterministically justified value of the slenderness limit of about 0.7 can be derived for these buckling curves.

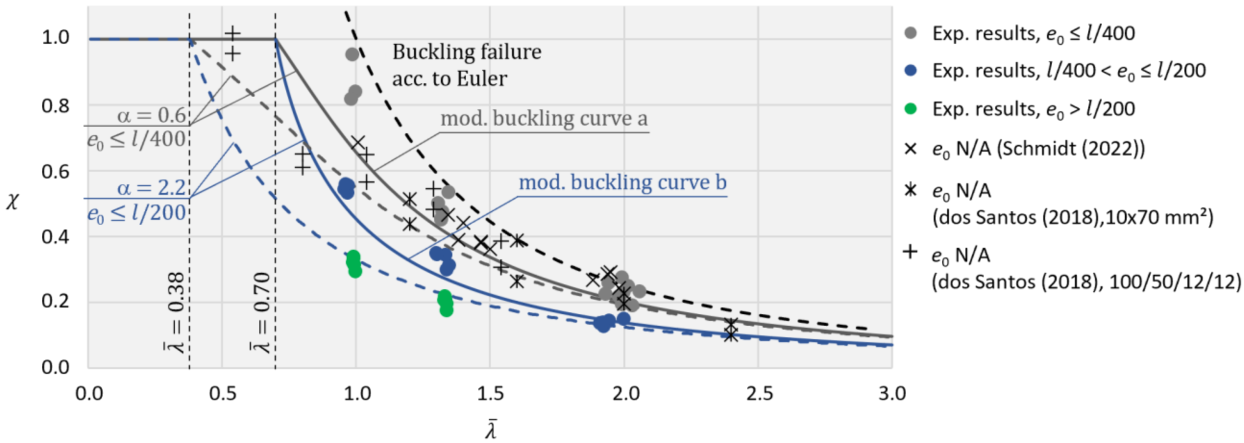

Figure 3: Modified buckling curves and test results from [23], [24, 25] and [26]

Both the results of tests with known eccentricity, which can be expressed as an equivalent initial bow imperfection $e_0 \leq l/400$ of a compression member with eccentricity e (see Fig. 4a), and the results of buckling tests from [24, 25] can be covered by the modified buckling curve a on the safe side. The modified buckling curve b is suitable to describe the limit state for the tests with equivalent initial bow imperfection $l/400 < e_0 \leq l/200$ according to [26] and the test results on rectangular compression members according to [23]. Due to the correlation shown in Eq. (5), the proposed set of buckling curves can only apply to a limited range of values for e_0. The behavior of compression members with geometric imperfections $e_0 > l/200$ and other structural imperfections, such as I-sections according to [23], could be described by further modified buckling curves. The comparison of the test results with the calculated results according to the presented methodology is shown in Fig. 4b.

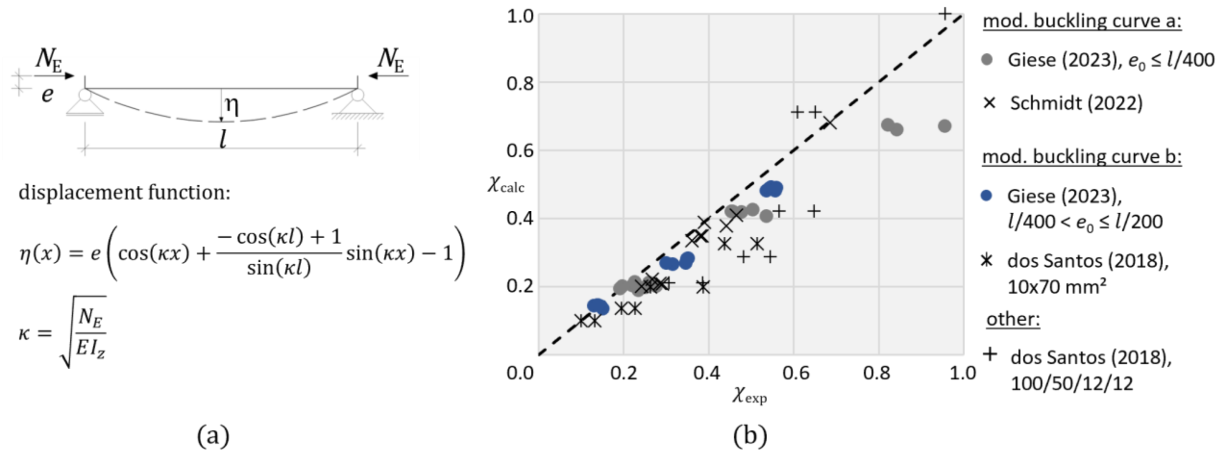

Figure 4: (a) model of compression member with load eccentricity; (b) comparison of test results and calculation

6 CONCLUSION

The advancement of slender CRC compression members has brought stability failure to the forefront of the design task, superseding material failure. At present, Eurocode 2 does not include any explicit regulations pertaining to CRC components. In a preliminary guideline for concrete components with non-metallic reinforcement, the design proposal is based on a calculation according to second-order theory. The present work demonstrates that the methodology of Eurocode 3 for steel construction can be applied to slender CRC compression members in principle.

In order to further develop the methodology, deeper considerations on the transition from material failure at small values for the slenderness to stability failure of slender CRC columns are required. This includes the determination of an appropriate limit for the slenderness λ or $\bar{\lambda}$. Furthermore, it is necessary to discuss which factors influence the shape of the buckling curves and thus the imperfection coefficient α. In addition to geometric imperfections such as load eccentricities and pre-curvatures, the scatter of the strength over the cross-section, the degree of reinforcement, the position of the reinforcement and the different distribution of the maximum grain sizes over the cross-section can play a decisive role. According to the procedure in EC3, such influencing variables could be understood as ‘structural imperfections’.

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