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Industry CPD

Composite column design

This CPD module, sponsored by SCIA, examines the analysis and design of steel-and-concrete composite columns.

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Composite columns, comprising steel and concrete components, present a range of benefits, including greater loadbearing capacity, improved stiffness, and enhanced fire resistance. When it comes to designing these columns, both EN 1994-1-1[1] and EN 1994-1-2[2] offer comprehensive guidelines, encompassing simplified methods. These simplified approaches provide engineers with a practical framework for analysis and design. This article dives into the subject to determine the most optimal method for analysing and designing composite columns.

When should composite columns be used?

The notion that composite columns reconcile the advantages of both steel and concrete while fully mitigating their drawbacks is a flawed assertion. A more accurate perspective would be that composite columns combine the benefits of steel and concrete while partially offsetting their inherent disadvantages. Moreover, this approach introduces additional constraints alongside its advantages.

Some examples of additional constraints are presented below:

- →| The required detailing constraints are important and could greatly impact the economy of the solution, especially for small cross-sections.
- →| There are different construction processes when comparing a composite column to a solely steel or a solely concrete column.
- →| Composite columns use construction techniques from both steel and concrete. As a consequence, different workmanship is involved in the process.

A composite column is the preferred solution where there is a combination of:

- →| concrete or steel-concrete flooring
- →| fire resistance requirements
- →| high slenderness of the steel section alone (concrete component will increase stiffness and reduce slenderness)
- →| high load capacity requirement (e.g. >10MN)
- →| eventual requirement of impact/blast resistance or robustness criteria
- →| seismic requirements.

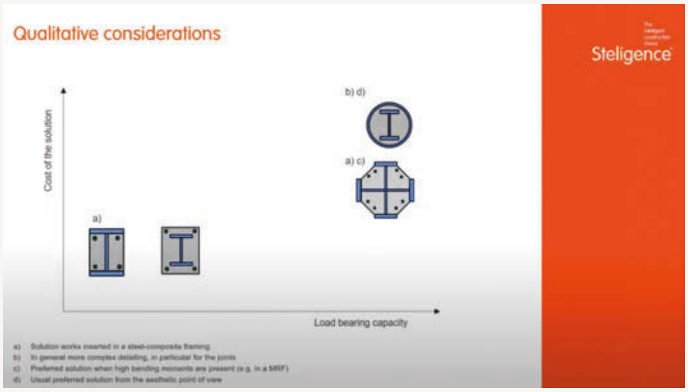


FIGURE 1: Qualitative considerations

For seismic requirements, the RFCS INERD project gave the following insights: steel profiles can mitigate 'soft storey' failure in reinforced concrete buildings. On average, composite columns which have steel profiles embedded within them: provide 2× more rotation capacity, resist 1.5× more cycles, and dissipate 3× more energy while demonstrating similar stiffness, thus keeping an analogous spectral response compared to reinforced concrete for the use cases considered within the study.

Based on the information extracted from **Figure 1**[3], it is evident that partially encased columns (such as an I-section with concrete between the flanges) are commonly employed for lower loadbearing capacities, offering a more cost-effective solution compared with fully encased alternatives. Partially encased composite columns are particularly effective when connected to steel beams, as their exposed flanges facilitate beam-to-column connections using methods like bolted connections.

The cost factor increases when opting for a cruciform composite column. However, the cruciform design presents an excellent solution for addressing high bending moments due to its geometry.

Composite columns comprising a tube enclosing a steel section, while offering a higher loadbearing capacity, generally entail higher costs due to increased detailing. Nonetheless, aesthetically, this configuration presents an attractive solution as only the circular shape of the steel tube is visible from the outside.

It is crucial to bear in mind that each composite column cross-section has its

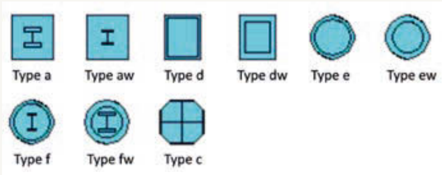


FIGURE 2: Simplified method – cross-section types

preferred field of application, as outlined based on its loadbearing capacity. Additionally, the complexity involved in constructing and assembling these columns on site affects the overall cost.

Simplified method of design

The design of composite columns involves considerations such as material properties, cross-sectional geometry, loadings, and stability. The simplified method, in accordance with EN 1994-1-1, streamlines the design process by assuming certain conditions and providing straightforward equations. These equations consider axial load, bending moment, and shear force distributions along the length of the column, leading to a more efficient and practical design approach.

The simplified method relies on several assumptions to ensure a conservative design, which implies that there are a set of constraints that need to be ensured, as mentioned in EN 1994-1-1 (mainly under art. 6.7.1, art. 6.7.3.1 and art. 6.7.5). An alternative is the general method of design in EN 1994-1-1, art. 6.7.2, which contains fewer constraints, allows any cross-section and can contain higher concrete grades than C50/60. However, this article focuses on the simplified method of design which is used in practice for >95% of the use cases[3].

Supported cross-sections

Various cross-section types are described within EN 1994-1-1 for the simplified method of design by means of figure 6.17, as illustrated in **Figure 2**[4].

These cross-sections can be categorised in the following groups, as mentioned in EN 1994-1-1, art. 6.7.1(1):

- concrete-encased cross-sections (type a)
- partially encased cross-sections (type a/c)
- concrete-filled rectangular and circular tubes (type d/e/f).

The general method of design as described in EN 1994-1-1, art. 6.7.2 can be used for other cross-sections that are not covered by the simplified method of design.

Composite column design

Within structural analysis software, a range of checks can be conducted to assess the capacity of a composite column under different structural loads. These checks are performed when the internal force is present for the specific combination being analysed. The subsequent paragraphs will outline the various checks that can be performed, starting with an explanation of how second-order effects are accounted for.

Second-order effects

Second-order effects induce additional bending moments due to the influence of structural deformations and/or imperfections.

An important distinction needs to be made between a linear analysis and a non-linear analysis with geometrical non-linearities **(Figure 3)**[5]. The selected path affects how the second-order effects are taken into account in the composite column code check. For further information, refer to figure 6.36 of Johnson and Anderson[6].

Non-linear analysis with geometrical non-linearity

This use case is described as a 'real second-order analysis' or a 'more accurate approach'. In this case, it is assumed that imperfections are defined through non-linear combinations and thus the second-order effects are, by replacing the stiffnesses EI of the composite columns in the analysis with the reduced stiffness $EI_{\text{eff,II}}$ (EN 1994-1-1, eq. 6.42), already included in the internal forces during the analysis phase.

Depending on the internal forces present for a given combination, further compression and bending moment resistances(s) are based on the interaction curve.

Linear analysis

If the previously mentioned non-linear analysis with geometrical non-linearity is not used, then the linear analysis approach is used. However, there is a distinction within this approach depending on whether bending moments are present.

Pure compression

If only compression forces are present (i.e. pure compression), possibly accompanied by shear forces (i.e. no bending moments), this is a pure compression use case. In such cases, there are two ways to verify pure compression: either via the simplified method or using European buckling curves as described in EN 1994-1-1, art. 6.7.3.5(2).

In the other case of pure compression (setting deactivated), an evaluation of the value of α for the y-y and z-z axis is needed, as described in EN 1994-1-1 art. 5.2.1(3):

$$\alpha = \frac{N_{\text{cr,eff}}}{|N_{\text{Ed}}|}$$

If $\alpha \geq 10$, second-order effects are not needed, and initial bending moments M_{Ed} are increased only by adding a member imperfection moment M_{imp}:

$$M_{\text{Ed,mod}} = M_{\text{Ed}} + M_{\text{ipm}}$$

If $\alpha < 10$, second-order effects are needed and are accounted for by increasing the initial bending moments via a member imperfection moment M_{imp} and applying multiplication factors k_1 and k_2:

$$M_{\text{Ed,mod}} = k_1 \cdot M_{\text{Ed}} + k_2 \cdot M_{\text{imp}}$$

The multiplication factors k_1 and k_2 are given by EN 1994-1-1, art. 6.7.3.4(5) in which k_2 deviates from that equation by replacing the moment factor β (EN 1994-1-1, Table 6.4) with 1.

Compression check

In general, there are two ways to evaluate a composite column member under compression:

1) Simplified method using European buckling curves (default): this method verifies the compression check by reducing the compression resistance $N_{\text{pl,Rd}}$ with the minimum reduction factor coming from χ_y or χ_z:

$$\frac{|N_{\text{Ed}}|}{\text{MIN}(\chi_y, \chi_z) \cdot N_{\text{pl,Rd}}} \leq 1$$

2) Compression check as a section check in combination with the interaction curve: this method verifies the compression check by fully using the compression resistance $N_{\text{pl,Rd}}$ as a section check:

$$\frac{|N_{Ed}|}{N_{\text{pl,Rd}}} \leq 1$$

Afterwards, the influence of the normal force on the bending moment is accounted for via the interaction curve.

Compression resistance

Depending on the chosen composite column cross-section, EN 1994-1-1, art. 6.7.3.2(1) offers formulas for calculating the compression resistance $N_{\text{pl,Rd}}$:

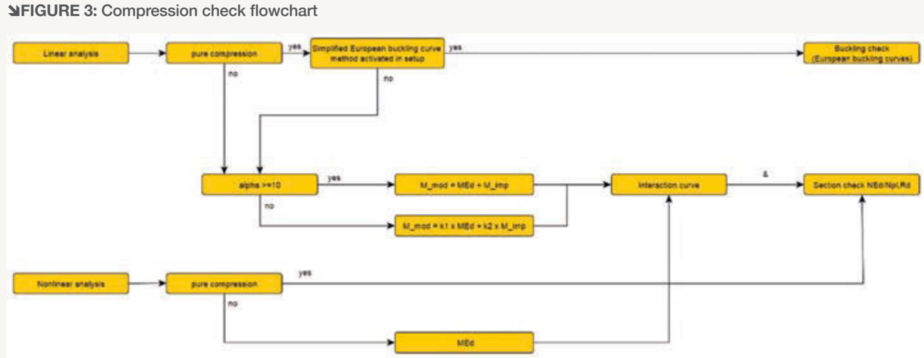
1) Concrete-encased (type a) and partially concrete-encased (type b, c & d):

$$N_{\text{pl,Rd}} = A_a \cdot f_{yd} + 0.85 \cdot A_c \cdot f_{cd} + A_s \cdot f_{sd}$$

2) Concrete-filled tube (type e & f):

$$N_{\text{pl,Rd}} = A_a \cdot f_{yd} + 1.00 \cdot A_c \cdot f_{cd} + A_s \cdot f_{sd}$$

FIGURE 3: Compression check flowchart



An additional strength can be gained for concrete-filled tubes through confinement if both conditions of EN 1994-1-1, art. 6.7.3.2 (6) are fulfilled. When fulfilled, the compression resistance is no longer determined by the above equation, instead equation 6.33 of EN 1994-1-1 is used:

$$N_{\text{pl,Rd}} = \eta_{\text{a}} \cdot A_{\text{a}} \cdot f_{\text{yd}} + 1.00 \cdot A_{\text{c}} \cdot f_{\text{cc}} + A_{\text{s}} \cdot f_{\text{sd}}$$

Transverse shear check

The transverse shear check is executed according to EN 1994-1-1, art. 6.2.2.2(1-2), with only the resistance of the structural steel ($V_{\text{pl,a,Rd}}$) considered. A contribution from the reinforced concrete part of the beam is not assumed to be established:

$$\frac{|V_{\text{Ed}}|}{V_{\text{pl,a,Rd}}} \leq 1$$

Longitudinal shear check

As given by EN 1994-1-1, art. 6.7.4.2 (2), the column length should be split into three areas:

Areas of load introduction with length lvi not exceeding $2d$ or $Ly/3$, where d is the minimum transverse dimension of the column cross-section and L is the span length. These areas are located at the start and end of the column.

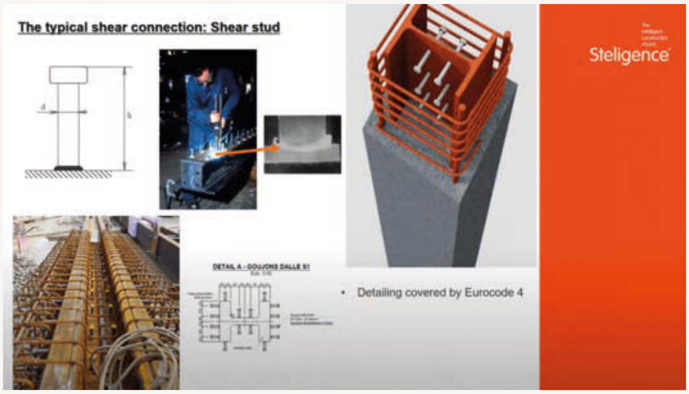
Area outside of load introduction which lies between the two areas from above (lvo = L – 2*lvi). As given by EN 1994-1-1, art. 6.7.4.1 (3), for axially loaded columns and compression members (= pure compression), longitudinal shear outside the areas of load introduction need not be considered.

For both inside and outside areas, longitudinal shear at the interface between concrete and steel is verified. Shear connectors should be provided, based on the distribution of the design value of longitudinal shear τ_{Ed}, where this exceeds the design shear strength τ_{Rd}. Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in EN 1994-1-1, Table 6.6 are assumed for τ_{Rd}:

$$\frac{\tau_{\text{Ed}}}{\tau_{\text{Rd}}} \leq 1$$

If conditions given by EN 1994-1-1, art. 6.7.4.3 (4) for fully concrete-encased steel sections (type a) are fulfilled, initial resistance τ_{Rd} may be increased by coefficient β_{c} (6.49) and the verification formula is then modified to:

$$\frac{\tau_{\text{Ed}}}{\beta_{\text{c}} \cdot \tau_{\text{Rd}}} \leq 1$$



↗FIGURE 4: Shear stud connection

Design longitudinal shear stress within area of load introduction

The design longitudinal shear stress τ_{Ed} is calculated using a formula initially based on equation 5.43 of Johnson[7], but in SCIA Engineer[4] it has been extended to also consider the effects from the bending moment around the y-axis:

$$\tau Ed = \frac{|N_{\text{c+s,Ed}}|}{\mu_{\text{a}} \cdot l_{\text{v}}} + \frac{|M_{\text{c+s,y,Ed}}| / (z \cdot z_{\text{coeff}})}{0.5 \cdot \mu_{\text{a}} \cdot l_{\text{v}}}$$

Headed studs check

As indicated, if $\tau_{\text{Ed}} > \tau_{\text{Rd}}$ then additional reinforcement by means of headed studs **(Figure 4)**[3] is needed. The check of headed studs is only supported for fully or partially encased cross-sections of type a and b.

The resistance of the headed stud is given by EN 1994-1-1, art. 6.6.3.1 as the smaller value of resistance $P_{\text{Rd,1}}$ (equation 6.18) and resistance $P_{\text{Rd,2}}$ (equation 6.19).

As given by EN 1994-1-1, art. 6.7.4.2 (4), the resistance of each horizontal row of studs may be increased by frictional forces present on the flanges, while fulfilling the geometry limits. Shear resistance of a horizontal row is then calculated as:

$$P_{\text{Rd,row}} = \eta_{\text{row}} \cdot \min(P_{\text{Rd,1}}, P_{\text{Rd,2}}) + \mu \cdot \min(P_{\text{Rd,1}}, P_{\text{Rd,2}})$$

The final resistance of the headed stud is calculated as:

$$P_{\text{Rd}} = \frac{P_{\text{Rd,row}}}{\eta_{\text{row}}}$$

The acting maximum design shear force is determined based on elastic and plastic theory:

Maximum design shear force $P_{\text{Ed,el}}$ according to elastic theory:

$$P_{\text{Ed,el}} = \sqrt{\left[\frac{N_{\text{c+s,Ed}}}{n} + \frac{M_{\text{c+s,y,Ed}}}{\sum r_{\text{i}}^2} \cdot x_{\text{i}}\right]^2 + \left[\frac{M_{\text{c+s,y,Ed}}}{\sum r_{\text{i}}^2} \cdot z_{\text{i}}\right]^2}$$

Maximum design shear force $P_{\text{Ed,pl}}$ according to plastic theory:

$$P_{\text{Ed,pl}} = \frac{N_{\text{c+s,Ed}}}{n} + \frac{M_{\text{c+s,y,Ed}}}{e_{\text{h}} \cdot 0.5 \cdot n}$$

The final verification of the headed studs is verified using:

$$\frac{\max(P_{\text{Ed,el}}; P_{\text{Ed,pl}})}{P_{\text{Rd}}} \leq 1$$

Combined compression and bending check

Interaction curve

The interaction curve is determined numerically in order to determine the capacity of the composite cross-section with regards to an interaction of internal forces ($N_{\text{Ed}} + M_{\text{Ed}}$). Depending on the load, the position of the neutral axis is changed and this leads to different values of compressive and tensile areas being obtained in composite members. Therefore, this results in a different capacity calculated from the strain distribution.

EN 1994-1-1 simplifies the interaction curve **(Figure 5)**[4] by means of calculating four points (A-B-C-D) and linearly interpolating the points between them.

This leads to less capacity between those four points as it is an approximation, and thus to a lower resistance of the composite cross-section. Software such as SCIA Engineer[4] are not limited to those four points and do calculate the entire curve, thus leading to a more exact and economical design.

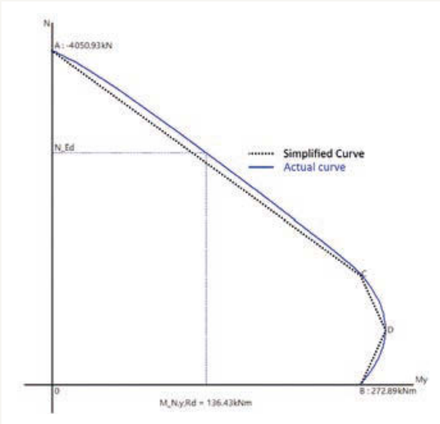
Influence of high shear force

If a high shear force is present ($V_{\text{Ed}} > 0.5{*}V_{\text{pl,Rd}}$), the yield strength of the structural steel section is reduced by means of the ρ-factor (EN 1994-1-1, equation 6.5).

This in turn influences the interaction curve as the structural steel (full section) uses a reduced yield strength. If the high shear force

Table 1: Tabulated approach to fire resistance (according to EN 1994-1-2)

Geometry criteria	Min. value	Actual value	Unit
1.1 Dimension h_c and b_c	150.00	159.00	mm
1.2 Concrete cover of steel section c	40.00	45.00	mm
1.3 Axis distance of reinforcing bars u_s	20	48	mm



↗FIGURE 5: Interaction curve for combined compression and bending

surpasses the shear resistance, leading to a failing transverse shear check, it also impacts the interaction curve.

<u>Internal forces that fall outside the interaction curve</u>
If the internal forces set (N_{Ed} + $M_{y,Ed}$ or N_{Ed} + $M_{z,Ed}$) fall outside the interaction curve, no combined check can be executed because these internal forces surpass the cross-section's capacity.

<u>Concrete-filled tubes of circular cross-section</u>
Account is taken of the increase of concrete strength of concrete-filled tubes of circular cross-section if they fulfil both criteria mentioned in EN 1994-1-1, art. 6.7.3.2(6).

The interaction curve in such cases is determined by an increase of f_{cd} due to multiplication by a factor:

$$\left(1+\eta_c \cdot \frac{t}{d} \cdot \frac{f_y}{f_{ck}}\right)$$

Combined compression and uniaxial bending
The combined compression and uniaxial bending check is executed according to EN 1994-1-1, art. 6.7.3.6:

$$\frac{M_{Ed}}{M_{pl,N,Rd}} = \frac{M_{Ed}}{\mu_d \cdot M_{pl,Rd}} \leq \alpha_M$$

Combined compression and biaxial bending
The combined compression and biaxial bending check is executed according to EN 1994-1-1, art. 6.7.3.7:

$$\frac{M_{y,Ed}}{\mu_{dy} \cdot M_{pl,y,Rd}} + \frac{M_{z,Ed}}{\mu_{dz} \cdot M_{pl,z,Rd}} \leq 1.0$$

Fire design situation

To assess the structural behaviour of composite columns in a fire design situation, EN 1994-1-2 defines the following design procedures:
→| tabulated data
→| simple calculation models
→| advanced calculation models.

Tabulated data and simple calculation models are applicable to specific types of structural members and provide more conservative results compared with advanced models. Their advantage lies in their ease of use to quickly assess the fire resistance of a member.

EN 1994-1-2, art. 4.2.3 provides tabulated data for all three standard types of composite column, i.e. concrete-encased sections, partially encased sections and concrete-filled hollow sections. In each case, minimal dimensions are given for the requested standard fire resistance **(Table 1)**[4].

Structural design software such as SCIA Engineer[4] and A3C[8] offer this tabulated approach for an easy assessment of composite columns in a fire design situation.

In art. 4.3.5 of EN 1994-1-2, simplified models are given to determine the compression resistance of a composite column under fire conditions. For partially encased and concrete-filled hollow sections, further reference is made to Annex G and H respectively.

The simplified models typically consist of a component-based approach where the resistance of each component (steel, concrete, reinforcement) is first determined individually and then combined to determine the resistance of the entire section.

When no simplified model is available, as is the case for a concrete-filled hollow section including a steel profile, an advanced calculation model is required. Such models typically consist of a numerical finite-element analysis of the section.

Within A3C[8], both simplified and advanced design models are available which allow the assessment of the fire design situation beyond the scope of the tabulated data.

Conclusion

This article has examined the simplified method for designing composite columns, shedding light on its intricacies. In addition to the simplified method, EN 1994-1-1 also presents the general method of design outlined in art. 6.7.2. This alternative approach allows for the utilisation of any composite cross-section and even permits the combination of various materials. However, the general method necessitates a numerical analysis of the cross-section and demands considerable expertise from the engineer, particularly when assessing second-order effects.

The general method proves valuable in special cases where the simplified method fails to provide an optimal solution or when a higher concrete strength exceeding C50/60 is required. Nevertheless, it is worth noting that in 95% of practical use cases, the simplified method of design has been employed and proven to be sufficient[3]. For those seeking to validate composite columns using the simplified method, structural design software such as SCIA Engineer[4] offers a solution that also takes into account second-order effects.

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