Design models for predicting the resistance of headed studs in profiled sheeting

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Abstract. This paper presents the results from reliability analyses of the current Eurocode 4 (EN 1994-1-1) and AISC 360-16 design models for predicting the resistance of headed stud shear connectors within profiled steel sheeting, when the ribs are oriented transverse to the supporting beam. For comparison purposes, the performance of the alternative "Luxembourg" and "Stuttgart" model were also considered. From an initial database of 611 push-out tests, 269 cases were included in the study, which ensured that the results were valid over a wide range of geometrical and material properties. It was found that the current EN 1994-1-1 design rules deliver a corrected partial safety factor γ_M^* of around 2.0, which is significantly higher than the target value 1.25. Moreover, 179 tests fell within the domain of the concrete-related failure design equation. Notwithstanding this, the EN 1994-1-1 equations provide satisfactory results for re-entrant profiled sheeting. The AISC 360-16 design equation for steel failure covers 263 of the tests in the database and delivers $\gamma_M^*\approx 2.0$. Conversely, whilst the alternative "Stuttgart" model provides an improvement over the current codes, only a corrected partial safety factor of $\gamma_M^*=1.47$ is achieved. Finally, the alternative "Luxembourg" design model was found to deliver the required target value, with a corrected partial safety factor γ_M^* between 1.21 and 1.28. Given the fact that the Luxembourg design model is the only model that achieved the target values required by EN 1990, it is recommended as a potential candidate for inclusion within the second generation of Eurocodes.

Keywords: Composite steel and concrete structures, Headed studs, Shear resistance, Partial safety factors, Eurocode 4, Reliability, Design models, Statistical evaluation.

1. Introduction

In steel-concrete composite beams, the transfer of longitudinal shear forces is crucial to guarantee the composite action of the members and is commonly achieved by the provision of mechanical shear connectors. Owing to the development of the electric drawn arc stud welding apparatus, the headed stud became the most used connector type in composite steel-concrete construction. The structural performance of this connector in solid slabs and encasements has been thoroughly studied in numerous push-out and full-scale beam tests starting from the 1950s (Thürlimann 1959; Viest 1956). In the subsequent years, innovative composite floor systems with profiled metal decking were also developed to optimize the design of buildings. The metal deck is typically composed of cold-formed steel sheeting with the ribs oriented parallel or perpendicular to the span of the supporting beams. The latter solution is generally preferred by construction industry because the profiled sheeting significantly increases the bending resistance of the slab. To support the application of this structural solution, numerous experimental tests were carried out in 1970's (Grant *et al.* 1977; Iyengar and Zils 1973; Fisher 1970)

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and 1980's (Roik and Bürkner 1981; Jayas and Hosain 1987), which were accompanied with the development of semi-empirical equations for predicting the resistance of studs in profiled sheeting. These tests formed the basis for the development of the design equations provided in the current EN 1994-1-1 (2004) and the North American AISC 360-16 (2016).

In the last decade trapezoidal, or 'open-trough' profiled steel sheeting, with slender ribs have increasingly studied (Nellinger *et al.* 2017; Sun *et al.* 2019; Russell *et al.* 2021) and employed worldwide, which reduces concrete volume and increases composite slab efficiency. Whilst these modern profiled steel sheets satisfy the geometric limitations defined by existing design codes, several studies (Vigneri 2021; Bonilla *et al.* 2018; Hicks 2007; Ernst *et al.* 2007; Odenbreit and Nellinger 2017) have shown that the EN 1994-1-1 (2004) design equations for estimating the resistance of studs in profiled sheeting provide overoptimistic predictions and do not always fulfil the safety requirements of the semi-probabilistic partial safety concept given in EN 1990 (2002). To remedy this situation, alternative design models have been proposed over the last two decades which may better describe the resistance of such connectors. In the framework of the European Commission's mandate M/515 (2012) for the development of the second generation of the Eurocodes, the committee responsible for EN 1994 (CEN/TC250/SC4) developed the task entitled 'Development of revised rules in EN 1994-1-1 for shear connection in the presence of modern forms of profiled sheeting' and assigned Project Team CEN/TC250/SC4.T3 the responsibility to recommend an alternative design model for implementation within the second generation of this standard.

By considering a database of push tests that is much larger than was considered in the original calibration of EN 1994-1-1 (n = 611 cf. n = 57 tests), the present paper considers the performance of the current EN 1994-1-1 and AISC 360-16 design models, before investigating the performance of two new design models proposed by the University of Luxembourg and the University of Stuttgart (hereafter referred to as "Luxembourg" and "Stuttgart", respectively). The resistance functions for these 4 design models are presented in Section 2, while the representative push test database considered is described in Section 3. The reliability analyses have been performed in accordance with the standard procedure for resistance models given in EN 1990 Annex D.8, given in Section 4. Finally, the results of the statistical evaluation are summarized and discussed in Section 5 and 6, respectively.

2. Design models

2.1. Current Eurocode 4 (EN 1994-1-1) design rules

From the results of push-out tests carried out in the 1970s, the first empirical equations for the resistance of shear stud connections were developed accounting for the negative effect of the profiled sheeting on the resistance of the connector. From preliminary statistical evaluations of the available test results, the resistance function proposed for predicting the resistance of studs in solid slabs (Ollgaard *et al.* 1971) was adapted through a reduction factor k_t (Grant *et al.* 1977). This coefficient is strongly dependent on the height of the connector and the geometry of the rib, as well as on the number of studs that are present. Following an increased number of test results becoming available in the next decade, this initial design proposal was adjusted and modified by taking into account the influence of other parameters, such as the thickness of the sheeting as well as the through-deck welding procedure (Bode and Künzel 1990; Johnson and Dongjie 1995; Stark and van Hove 1991). The resulting resistance function r_t given in current EN 1994-1-1 for calculating the resistance of studs placed in profiled steel sheeting transverse to the supporting beam is defined as follows:

$$r_t = min\{r_{t,s}, r_{t,c}\} \tag{1}$$

and

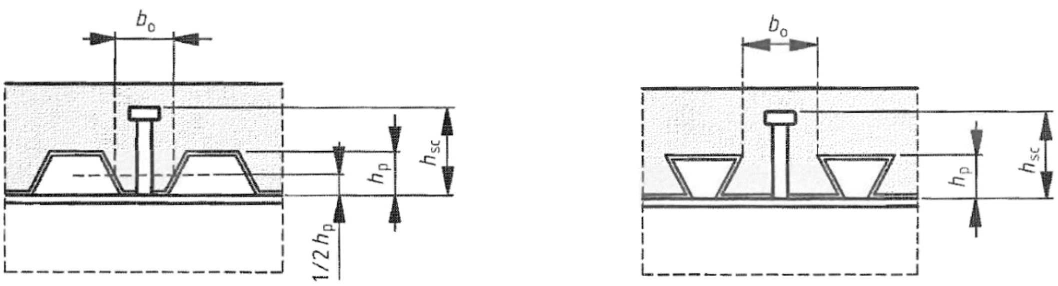
$$r_{t,s} = 0.80 \cdot k_t \cdot f_u \pi \frac{d^2}{4} \tag{2}$$

$$r_{t,c} = 0.29 \cdot k_t \cdot d^2 \alpha \sqrt{f_c E_c} \tag{3}$$

With:

$$k_t = min\left\{\frac{0.7}{\sqrt{n_r}} \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right), k_{t,max}\right\} \tag{4}$$

$$\alpha = \begin{cases} 0.2\left(\frac{h_{sc}}{d} + 1\right) & for\ 3 \leq h_{sc}/d \leq 4 \\ 1 & for\ h_{sc}/d > 4 \end{cases} \tag{5}$$



(a) (b)

Fig. 1 Geometrical properties for (a) open-trough and (b) re-entrant profiled sheeting according to EN 1994-1-1 (2004) design model.

where:

d is the diameter of the shank of the stud

f_u is the ultimate tensile strength of the stud material

f_c is the cylinder compressive strength of the concrete of density not less than 1750 kg/m³

E_c is the short-term secant elastic modulus of the concrete, according to the EN 1992-1-1 (2004) relationship

h_{sc} is the height of the stud after welding

n_r is the number of stud connectors in one rib at the intersection with the beam

h_p is the overall depth of the sheeting excluding embossments

b_0 is the mean width of the concrete rib, as defined in Fig. 1.

Table 1 – Upper limits $k_{t,max}$ for the reduction factor k_t.

$k_{t,max}$	Thickness of the sheeting t	Studs not exceeding 20 mm in diameter and welded through profiled steel sheeting	Profiled sheeting with holes and studs 19mm or 22mm in diameter
n_r=1	t≤1 mm	0.85	0.75
	t>1 mm	1.0	0.75
n_r=2	t≤1 mm	0.70	0.60
	t>1 mm	0.80	0.60

2.2. Current AISC 360-16 design rules

The North American AISC 360-16 (2016) shares some analogies with the formulation given by EN 1994-1-1 and described in the previous section. In this case two additional reduction factors, namely R_g and R_p, were added only to the steel failure related equation for the resistance of studs within a solid slab. Whilst the former coefficient accounts for the effect of the number of studs per rib, the latter describes implicitly the impact of the eccentric position of the studs within the rib, which is dependent on the distance $e_{mid\text{-}ht}$, defined in Fig. 2. If this dimension is higher than 50 mm, the value of R_p should be further reduced from 0.75 to 0.6. However, no difference is considered between studs placed in pre-punched sheeting and through-deck welded studs. It should be noted that AISC 360-16 states that the application of this formulation is limited to studs with a length not less than four times the diameter of the shank.

The equation for predicting the resistance of studs according to the AISC 360-16 (2016) is given by the following expression:

$$r_t = min\{r_{t,s}, r_{t,c}\} \tag{6}$$

and

$$r_{t,s} = 1.00 \cdot R_g \cdot R_p \cdot f_u \pi \frac{d^2}{4} \tag{7}$$

$$r_{t,c} = 0.50 \cdot \pi \frac{d^2}{4} \sqrt{f_c E_c} \tag{8}$$

With:

$$R_g = \begin{cases} 1.0 & for\ n_r = 1 \\ 0.85 & for\ n_r = 2 \end{cases} \tag{9}$$

$$R_p = \begin{cases} 0.75 & for\ e_{mid-ht} \geq 50\ mm \\ 0.6 & otherwise \end{cases} \tag{10}$$

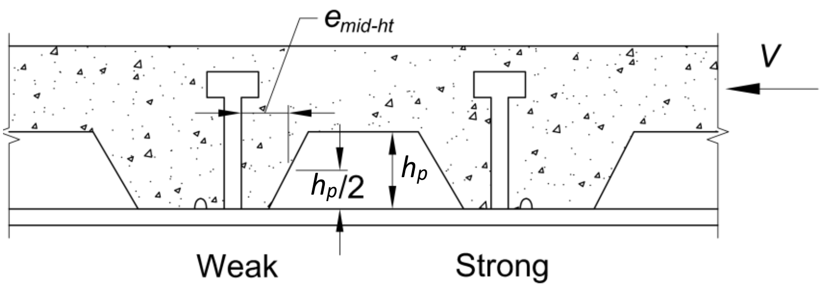


Fig. 2 Definition of the dimension $e_{mid\text{-}ht}$ according to AISC 360-16 (2016) design model.

Where:

E_c is the elastic modulus of the concrete, calculated according to AISC 360-16 (2016) relationship in Eq.(11)

w_c is the weight of the concrete per unit volume ($1500 \leq w_c \leq 2500$ kg/m^3). If no value was provided, w_c=2300 kg/m^3 was used in computation.

$$E_c = 0.043\, w_c^{1.5} \sqrt{f_c}\ [\mathrm{MPa}] \tag{11}$$

2.3. Luxembourg design model

The philosophy behind the "Luxembourg" design model differs significantly from the other approaches proposed in the last decades. In the framework of the European Commission's mandate M/515 (2012) for the development of the second generation of the Eurocodes, Project Team CEN/TC250/SC4.T3 were responsible for developing revised rules for EN 1994-1-1, to support the use of studs in modern forms of profile steel sheeting. Subsequently several statistical investigations were carried out to check the range within the current regulations were still applicable in fulfilment of the safety requirements given in EN 1990 (2002), whilst at the same time minimizing the negative impact of new regulations on the European construction industry. It was found that the main parameters affecting the suitability of EN 1994-1-1 are the embedment depth h_A=h_{sc}-h_p and the effective width of the rib e_k (calculated from the centre of the connector), defined in Fig. 3. Therefore, new design equations were proposed only within a limited range, as shown in the flow-chart in Fig. 4.

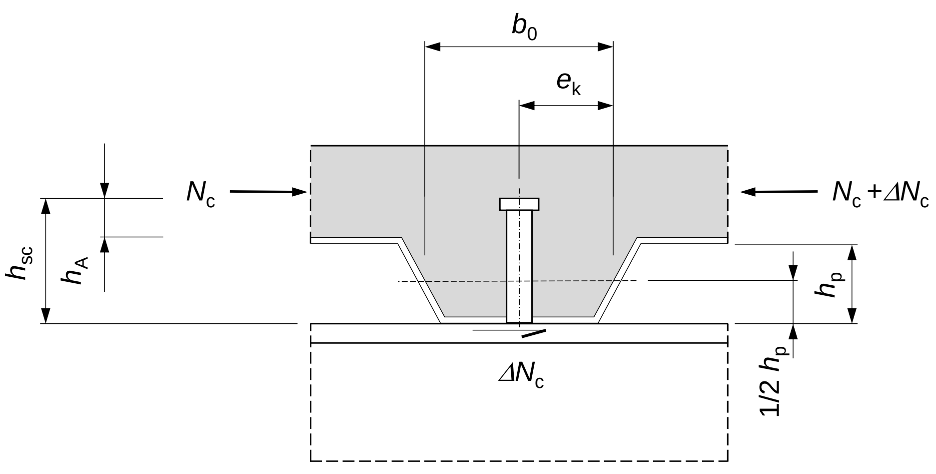


Fig. 3 Geometrical properties for open-trough profiled sheeting according to the "Luxembourg" design model.

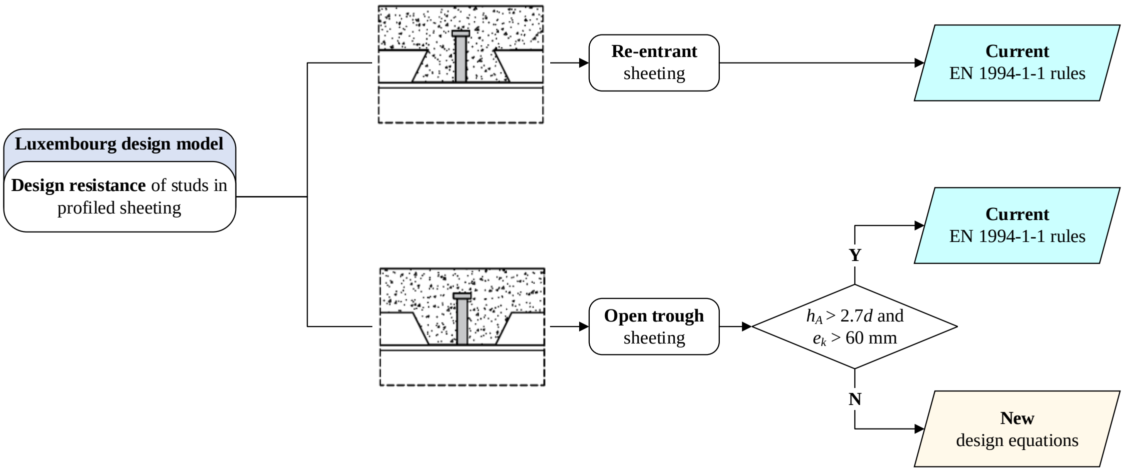


Fig. 4 Flow-chart of the "Luxembourg" design model for the shear resistance of studs in profiled steel sheeting.

According to preliminary statistical investigations, EN 1994-1-1 appears to be valid for all re-entrant profiles. In case of open trough profiled sheeting, if both conditions e_k>60 mm and h_A>2.7d are not fulfilled (as is common in modern open-trough profiled steel sheeting), the EN 1994-1-1 design rules are not applicable and new equations should be used. These new equations were developed in the last few years and are based on two resistance components acting in parallel: (i) "concrete cone" and (ii) "stud in bending". These components were identified in previous investigations (Lloyd and Wright 1990; Roik and Lungershausen 1989) and combined into one mechanical model by Nellinger (2015). This model was further improved by Vigneri (2021) with the support of experimental evidence and extensive numerical simulations (Vigneri *et al.* 2019a).

The resulting resistance given in Eq.(12) is composed of two equations. Whilst Eq.(13) considers only the failure of the stud shank, Eq. (14) defines the resistance of the shear connection owing to the combination of concrete cone failure and plastic bending deformation of the connector. It is worth noting that the correction factor k_u, defined in Table 2, accounts for the beneficial influence of the through-deck welding on the stud resistance, which is equal to 1.0 for thicknesses lower than 1 mm and when the stud is positioned centrally within the rib. Unlike previous studies (Stark and van Hove 1991; Roik *et al.* 1988), the values of k_u also account for the shorter length after welding for through-deck welded studs compared to the nominal length after welding given by ISO 13918 (2018), which is appropriate for studs with pre-punched holes.

$$r_t = min\{r_{t,s}, r_{t,c}\} \tag{12}$$

and:

$$r_{t,s} = 0.58 \cdot f_u \pi \frac{d^2}{4} \tag{13}$$

$$r_{t,c} = 1.0 \cdot C_2 \cdot k_u \cdot \left[\frac{f_{ct}\, W}{h_p\, n_r} + \frac{n_y\, f_u d^3/6}{0.82 h_p - d/2}\right] \tag{14}$$

With:

$$C_2 = 1.85\frac{h_p}{b_0} \leq 1.35 \quad \textit{but not smaller than } 1.0 \tag{15}$$

$$W = [2.4h_{sc} + (n_r - 1)e_t]\,\frac{b_{top}^2}{6} \tag{16}$$

$$n_y = \begin{cases} 2 & for\ \ n_r = 1\ or\ staggered\ position \\ 1 + \dfrac{h_A - 2d}{0.52d} \leq 2 & otherwise \end{cases} \tag{17}$$

Where:

h_p is the overall height of the profiled steel sheeting, excluding the re-entrant stiffener when its height is less than or equal to 15 mm

f_{ct} is the tensile strength of the concrete which can be calculated according to EN 1992-1-1 (2004) relationship given in Eq.(18)

e_t is the transverse spacing between the studs in the rib

$$f_{ct} = 0.3(f_c[MPa] - 8)^{2/3}\ \ [MPa] \qquad \text{for strength classes} \leq \text{C50/60} \tag{18}$$

Table 2 – Values of the correction factor k_u.

k_u	Profiled sheeting with pre-punched holes	Through-deck welded studs t < 1.0 mm	Through-deck welded studs $t \geq$ 1.0 mm
Centred or staggered position	1.0	1.0	1.25
Favourable position	1.1	1.1	1.38
Unfavourable position	0.8	0.8	1.0

2.4. Stuttgart design model

In 2010, researchers at the University of Stuttgart developed an empirical method for predicting the design resistance of stud connectors following the same approach as EN 1994-1-1 and AISC 360-16 (Konrad *et al.* 2020). Unlike the "Luxembourg" design model described in the previous section, this solution involves the replacement of current design rules not only for studs in profiled sheeting, but also for studs in solid slabs and encasements. The novelty of this approach lies in the fact that the newly developed expressions account for the effect of the weld collar dimensions on the stud resistance. The negative influence of the sheeting on the resistance of the shear connection is considered via the reduction factor $k_\perp$ which is applied only to one of the equations. This empirically-estimated coefficient considers the geometry of the deck, the relative position of the stud in the rib as well as the number of studs per rib and the type of welding.

The resulting resistance function proposed by "Stuttgart" is given by the following expression (Konrad *et al.* 2020):

$$r_t = min\{r_{t,s}, r_{t,c}\} \tag{19}$$

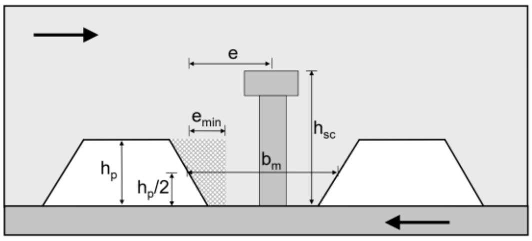
$$r_{t,s} = 313A_{col}\left(\frac{f_c}{30}\right)^{2/3} + 240d^2\left(\frac{f_u}{500}\right) \tag{20}$$

$$r_{t,c} = k_\perp \cdot \left[326A_{col}\left(\frac{f_c}{30}\right)^{2/3} + 220d^2\left(\frac{f_c}{30}\right)^{2/3}\left(\frac{f_u}{500}\right)^{1/2}\right] \tag{21}$$

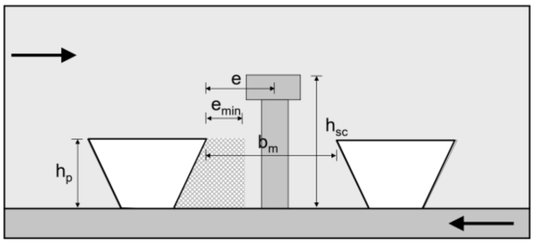
With:

$$A_{col} = 0.5d_{col}\,h_{col}\ \text{(Table 3)} \tag{22}$$

$$k_\perp = \begin{cases} k_n \cdot \left[0.038k_e\dfrac{b_m}{h_p} + 0.597\right] \leq 1.0 & \begin{cases}Pre\ punched\ sheeting; and\\ e \geq 55\ mm\end{cases} \\ k_n k_{Tr} \cdot \left[0.042k_e\dfrac{b_m}{h_p} + 0.663\right] \leq 1.0 & \begin{cases}Through\ deck\ welding; and\\ e \geq 55\ mm\end{cases} \\ k_n \cdot \left[0.317\dfrac{b_m}{h_p} + 0.06\right] \leq 0.8 & e < 55\ mm \end{cases} \tag{23}$$



(a)



(b)

Fig. 5 Geometrical properties for (a) open-trough and (b) re-entrant profiled sheeting according to the "Stuttgart" design model .

Table 3 – Effective weld collar area (Konrad *et al.* 2020).

Nominal shank diameter d_{nom} [mm]	*Height of the collar* h_{col} [mm]	*Diameter of the collar* d_{col} [mm]	*Effective area of the collar* A_{col} [mm^2]
16	4.5	21	47.3
19	6.0	23	69.0
22	6.0	29	87.0
25	7.0	31	108.5

3. Representative push-out test database

A comprehensive database of push tests on headed studs in profiled sheeting was established during the research project "DISCCO" between 2012 and 2015. This database was continuously updated within the University of Luxembourg project "ShearCON" (Vigneri 2021) between 2016 and 2020 in order to support project team CEN/TC250/SC4.PT3, who were responsible for developing revised rules for EN 1994-1-1. The creation of this database was also facilitated by the collaboration with several universities across the world and is freely available online (Vigneri *et al.* 2021). This database consists of a total 611 push-out tests on headed stud shear connectors in profiled sheeting transverse to the beam. A total of 269 tests were deemed to be representative of the conditions of the shear connection for the reliability analyses of the design models. These tests fulfilled the following conditions:

- No reported weld seam faults.
- No transversal load applied: As shown in recent experimental studies (Nellinger *et al.* 2017; Hicks and Smith 2014), the application of transversal load on the slab in push-out tests increases the resistance of the shear connection by more than 5%-10%. Although this condition might be representative of the real conditions of the beams, it is not yet clear whether the vertical load always applies at any point of the slab in building applications because local uplift might occur (Chapman 1964). Hence, the tests with the application of transversal load on the slabs were conservatively excluded from the present reliability study. On the other hand, recent push tests involving the interaction of the longitudinal shear and tension in studs were not included (Shen and Chung 2017).
- Not more than 2 studs per rib ($n_r \leq 2$).
- No lightweight concrete.
- Mean compressive cylinder strength of concrete f_{cm} not lower than 24 MPa.

Notwithstanding that older tests may have included the short-term concrete relaxation effects in the resistance values provided, these effects are not considered for the determination of the experimental value of the resistance wherever possible (Hanswille *et al.* 2007). For re-entrant sheeting with studs in the unfavourable position, the only available test showed a poor load bearing capacity. Due to the lack of further data, this push-out test was excluded. Apart from this exception, the database includes tests with studs in (i) centred, (ii) favourable, staggered and (iii) unfavourable position, Fig. 6.

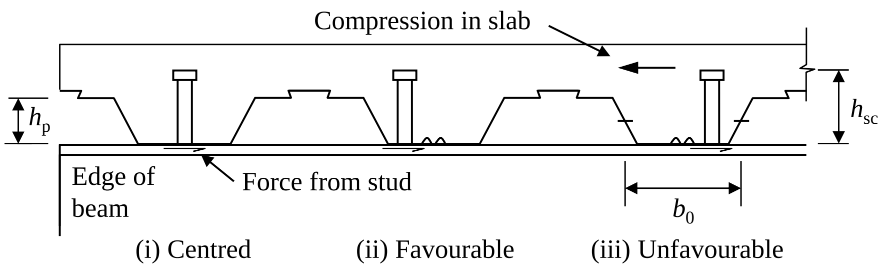


Fig. 6 Definition of headed stud position.

The objective of this investigation is to have a consistent comparison among different design models and to assess whether they can be applied even for modern and more slender profiled sheeting, knowing that the limitations given by current EN 1994-1-1 are exceeded in the present evaluation. It should be mentioned that these boundaries are based on the original calibration in which only 57 push-out tests were used. The 269 representative tests considered for the reliability analyses presented in the next sections cover a wide range of geometrical and mechanical variables, as shown in Table 4.

Table 4 – Limits of the basic variables X_i in the representative test database considered.

Property	*Min*	*Max*
(Nominal) diameter of the stud - d_{nom}^{*}	19 mm	22 mm
Mean as-welded length of the stud - h_{scm}	70 mm	200 mm
(Nominal) net height of the deck – h_p	40 mm	136 mm
(Nominal) bottom width of the deck - b_{bot}	40 mm	160 mm
(Nominal) top width of the deck – b_{top}	101 mm	240 mm
(Nominal) thickness of the sheeting – t	0.6 mm	1.2 mm
Mean compressive cylinder strength of concrete - f_{cm}	24 MPa	58.1 MPa
Mean ultimate tensile strength of stud material - f_{um}	417 MPa	570 MPa

*For through-deck welded studs, the database contains only studs with diameter not exceeding 19 mm

4. Design assisted by testing for calibrating resistance models on the basis of EN 1990

EN 1990, Annex D.8 (2002) provides a guideline to derive the design resistance from the statistical evaluation of the theoretical resistance function against representative test results. All the steps of the procedure and all the variables needed for the reliability analyses of the resistance functions defined in Section 2 are given in this section.

- *Step 1: Definition of the resistance function*

The theoretical resistance of the model r_t is represented by the analytical function $g_{rt}(\underline{X})$ covering all the relevant independent mechanical and geometrical variables $\underline{X}$ (namely, basic variables)**:**

$$r_t(\underline{X}) = g_{rt}(\underline{X}) \tag{24}$$

- *Step 2: Comparison between experimental and theoretical values*

To preliminary check the suitability of the design model, the theoretical resistance function using mean values $g_{rt}(\underline{X}_m)$ is compared with experimental resistance values r_e. The mean values of the mechanical properties were taken as mean measured values. If the geometrical dimensions were not available, the mean values of geometrical properties were calculated as a function of the nominal values. A detailed list of the mean values of the basic variables are shown in Table 5 including the values of the coefficient of variation.

The suitability of the design model should be firstly checked by calculating the coefficient of correlation ρ estimated according to Eq.(25). If this the value of the correlation factor is sufficiently high, the design model can be considered appropriate.

$$\rho = \frac{\sum_{i=1}^{n} r_{e,i} r_{t,i} - n\, \overline{r_e}\,\overline{r_t}}{(n-1)\sigma_{r_e}\sigma_{r_t}} \tag{25}$$

- *Step 3: Estimate the mean value correction factor b*

The probabilistic model of the resistance r can be represented by

$$r = b r_t \delta \tag{26}$$

Where δ is the error term, and b is the mean correction factor calculated as the "Least Squares" best-fit to the slope:

$$b = \frac{\sum_{i=1}^{n} r_{e,i} r_{t,i}}{\sum_{i=1}^{n} {r_{t,i}}^2} \tag{27}$$

With:

$$\overline{r_e} = \frac{1}{n}\sum_{i=1}^{n} r_{e,i} \tag{28}$$

$$\overline{r_t} = \frac{1}{n}\sum_{i=1}^{n} r_{t,i} \tag{29}$$

And:

$$\sigma_{r_e} = \frac{1}{n-1}\left(\sum_{i=1}^{n} {r_{e,i}}^2 - n\,\overline{r_e}^2\right) \tag{30}$$

$$\sigma_{r_t} = \frac{1}{n-1}\left(\sum_{i=1}^{n} {r_{t,i}}^2 - n\,\overline{r_t}^2\right) \tag{31}$$

- *Step 4: Estimate the coefficient of variation of the errors V_δ*

The error term δ_i for each experimental value $r_{e,i}$ is determined from the following expression:

$$\delta_i = \frac{r_{e,i}}{b r_{t,i}} \tag{32}$$

To estimate the value of the coefficient of variation V_δ of the error terms δ_i which are assumed to be log-normally distributed, the parameter Δ_i should be firstly defined:

$$\Delta_i = ln\,\delta_i \tag{33}$$

The estimated value of the mean for the expected value of Δ can be determined as follows:

$$\overline{\Delta} = \frac{1}{n}\sum_{i=1}^{n} \Delta_i \tag{34}$$

Therefore, the variance of the parameter Δ can be calculated from:

$$s_\Delta^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left(\Delta_i - \overline{\Delta}\right)^2 \tag{35}$$

Finally, the following expression can be used to determine the coefficient of variation of the error terms V_δ:

$$V_\delta^2 = e^{s_\Delta^2} - 1 \tag{36}$$

- *Step 5: Compatibility analysis*

To determine which parameters have most influence on the scatter, the test results may be split into subsets with respect to these parameters. The fractile factors were conservatively determined on the basis of the number of the tests in each subset because the field of application of the resistance function considered was not defined yet.

- *Step 6: Coefficients of variation of the basic variables V_{Xi}*

From the push-out test database, there is not sufficient information about the actual variability of the basic variables X_i. Therefore, the coefficient of variation V_{Xi} of these parameters shall be determined by prior knowledge. The mean value and the coefficient of variation for each basic variable were chosen according to several scientific sources, which are given in Table 5. In accordance with EN 1990, all the basic variables are assumed to be normally distributed. For some products, different tolerances were given. For these cases, the highest value of the coefficient of variation was used in the reliability analysis.

Table 5 – Mean values of the coefficient of variation of the basic variables X_i.

	Basic variables X_i	*Mean value X_m*	*Coefficient of variation V_X*	*References*
Mechanical Properties	f_c	$f_{c,meas}$	$8/1.64f_{cm}$[MPa]	EN 1992-1-1 (2004), fib (2013)
	f_u	$f_{u,meas}$	0.05	Roik *et al.* (1988)
Geometrical Properties	d	$d_{nom} - 0.2$ [mm]	$0.2/1.64d_m$[mm]	Hicks (2017), fib (2013); ISO 13918 (2018)
	h_{sc}	$h_{sc,meas}$	0.01	Nellinger (2015)
	h_p	$h_{p,nom}$	$2/1.64h_{pm}$[mm]	EN 1090-4 (2018)
	b_{top}	$b_{top,nom}$	0.05	Nellinger (2015), EN 1090-4 (2018)
	b_{bot}	$b_{bot,nom}$	0.05	Nellinger (2015), EN 1090-4 (2018)
	e_t	$e_{t,nom}$	0.10	Nellinger (2015)
	h_{col}	$h_{col,nom}$	0.20	Döinghaus (2001)
	d_{col}	$d_{col,nom}$	0.20	Döinghaus (2001)

- *Step 7: Characteristic value of the resistance r_k*

If V_δ and $V_{X,i}$ are relatively small, the following approximation can be used to determine the coefficient of variation of the resistance function V_r:

$$V_r^2 = V_\delta^2 + V_{rt}^2 \tag{37}$$

V_{rt} is the coefficient of variation of the theoretical resistance r_t from uncertainties in the basic variables X. Owing to the non-differentiability of the resistance functions r_t in their whole domain, the coefficient of variation V_{rt} was estimated by randomly extracting values of the basic variables that fell within the distributions defined by the data in Table 5. This computational method is known as "Monte-Carlo simulations". The characteristic resistance r_k should be calculated from the following expression:

$$r_k = b\ g_{rt}(\underline{X}_m)\ e^{(-k_\infty \alpha_{rt} Q_{rt} - k_n \alpha_\delta Q_\delta - 0.5Q^2)} \tag{38}$$

The characteristic resistance r_k should be calculated from the following expression:

$$Q_\delta = \sqrt{ln(V_\delta^2 + 1)} \tag{39}$$

$$Q_{rt} = \sqrt{ln(V_{rt}^2 + 1)} \tag{40}$$

$$Q = \sqrt{ln(V_r^2 + 1)} \tag{41}$$

$$\alpha_\delta = \frac{Q_\delta}{Q} \tag{42}$$

$$\alpha_{rt} = \frac{Q_{rt}}{Q} \tag{43}$$

The fractile factors *k* were calculated for an unknown coefficient of variation. These values are based on a t-distribution and the fractile factor *k(p)* for a probability *p* is determined in accordance with ISO 12491 (1997) by:

$$k(p) = t_p(v) \cdot \left(1 + \frac{1}{n}\right)^{0.5} \tag{44}$$

Where $t_p(\nu)$ is the fractile of the *t*-distribution for the probability *p* and number of degrees of freedom $v=n-1$. The characteristic k_n and design fractile factor $k_{d,n}$ were calculated for $p=5\%$ and $p=0.1\%$, respectively.

- *Step 8: Design value of the resistance r_d*

The design resistance r_d can be calculated from the following expression:

$$r_d = b\ g_{rt}(\underline{X}_m)\ e^{(-k_{d,\infty}\alpha_{rt}Q_{rt}-k_{d,n}\alpha_\delta Q_\delta-0.5Q^2)} \tag{45}$$

Where $k_{d,n}$ and $k_{d,\infty}$ are the fractile factors of a normal distribution corresponding to a cumulative probability of 0.1% for a finite and infinite sample size (for the latter case $k_{d,\infty} = \alpha_R\beta$, where α_R is the FORM sensitivity factor for resistance with a value of 0.8 and β is the reliability index with a value of 3.8 for a 50-year reference period), respectively. The partial safety factor γ_M is given by:

$$\gamma_M = \frac{r_k}{r_d} \tag{46}$$

- *Step 9: Corrected partial safety factor γ_M^**

In common design practice for steel and steel-concrete composite structures, the resulting expression for calculating the design resistance according to the resistance model is normally calculated from the nominal values of the basic variables, as opposed to characteristic values corresponding to a predefined fractile (often 5%) of the underlying statistical distribution of a material or geometric quantity (ENV 1993-1-1 1998, da Silva *et al.* 2017). Whilst the nominal values of the geometrical variables usually refer to the mean, the nominal strength of stud material does not correspond to the 5% fractile characteristic strength, as it corresponds to a guaranteed minimum value and thus often lies significantly below the 5% fractile (Roik *et al.* 1988). Therefore, the respective fractile factor k_∞ is higher than the value 1.64 used for determining the characteristic value of a certain property. As suggested by JCSS (2001) and applied in by Stark and van Hove (1991) and Hicks (2017), k_∞ was conservatively taken as

2.0 which corresponds to the 2.3% fractile. All the mean and nominal values of the basic variables are presented in Table 6.

Table 6 – Mean and nominal values of the basic variables X_i.

	Basic variables X_i	*Mean value* X_m	*Nominal value* X_n
Mechanical Properties	f_c	$f_{c,meas}$	f_{ck}
	f_u	$f_{u,meas}$	$f_{um}(1 - k_\infty V_{fu}) \leq f_{u,lim}^{**}$
Geometrical Properties	d	$d_{nom} - 0.2[\mathrm{mm}]$	d_{nom}
	h_{sc}	$h_{sc,meas}$	${l_2}^*$
	h_p	$h_{p,nom}$	$h_{p,nom}$
	b_{top}	$b_{top,nom}$	$b_{top,nom}$
	b_{bot}	$b_{bot,nom}$	$b_{bot,nom}$
	e_t	$e_{t,nom}$	$e_{t,nom}$
	h_{col}	$h_{col,nom}$	$h_{col,nom}$
	b_{col}	$b_{col,nom}$	$b_{col,nom}$

$^*l_2$ is the nominal as welded height of the stud according to ISO 13918 (2018)

$^{**}f_{u,lim}$ is 450 MPa in all design models presented except for the "Stuttgart" design model, where $f_{u,lim}$=500 MPa

Therefore, the partial safety factor γ_M defined in Eq.(46) should be adjusted by a factor k_c in order to account for the nominal resistance r_n. Finally, the average value of k_c is used for determining the corrected partial safety factor $\gamma_M{}^*$, defined as follows:

$$\gamma_M{}^* = k_c\,\gamma_M = \frac{r_n}{r_d} \tag{47}$$

- *Step 10: Final choice of the partial safety factor*

Based on the corrected partial safety factor obtained by the statistical procedure $\gamma_M{}^*$, the relevant class of partial safety factor γ_{Ri} is chosen and the final design resistance function is given by:

$$r_d = \frac{r_n}{\gamma_M{}^*} = r_n\left(\frac{\gamma_{Ri}}{\gamma_M{}^*}\right)\frac{1}{\gamma_{Ri}} \tag{48}$$

In the interest of harmonization with the design of other types of steel connections according to Eurocodes, where the failure is governed by fracture of bolts or welds, the recommended partial safety factor γ_{Ri} for resistance models for predicting the shear resistance of studs is given by γ_V=1.25 (EN 1993-1-1, 2004). Therefore, to ensure a good calibration of the design model, the corrected partial safety factor $\gamma_M{}^*$ should not differ significantly from the target value γ_V=1.25.

5. Statistical evaluation of the design models

5.1. Current Eurocode 4 (EN 1994-1-1) design rules

As can be seen in Fig. 7, both EN 1994-1-1 equations Eq.(2) and Eq.(3) deliver a required partial safety factor $\gamma_M{}^*$=1.916 and $\gamma_M{}^*$=2.159 for steel and concrete failure, respectively. Both values are significantly higher than the target value of 1.25 resulting in an overestimation of the design resistance of approximately 50-60%. The coefficient of variation V_r increases from 0.225 for steel failure up to

0.318 for concrete failure. However, this difference may be partially justified from the relatively higher variability of the concrete compressive strength. The statistical evaluations were also performed for different shapes of profiled sheeting, open trough and re-entrant, as shown in Fig. 8 and Fig. 9, respectively. Whilst the design resistance given by EN 1994-1-1 for studs in open trough sheeting is importantly larger than the target value, the application of the design equations leads to satisfactory results for the re-entrant sheeting considered in the database. Because these equations for predicting the resistance of studs are empirically derived, steel sheeting with new rib geometries should be carefully tested in order to check the suitability of the design rules.

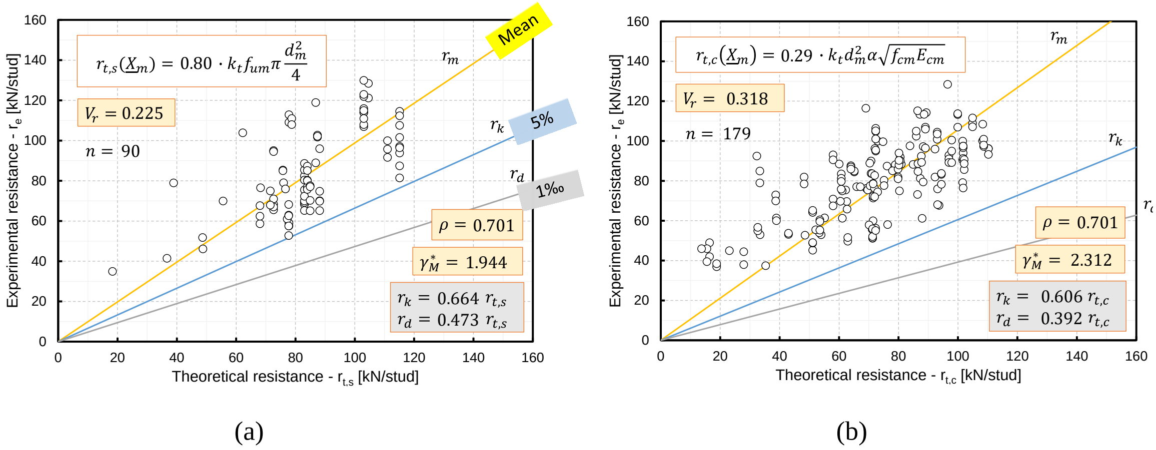


Fig. 7 Comparison between experimental and theoretical resistance according to EN 1994-1-1 design model (n=269 tests): (a) Eq.(2) for steel and (b) Eq.(3) for concrete failure.

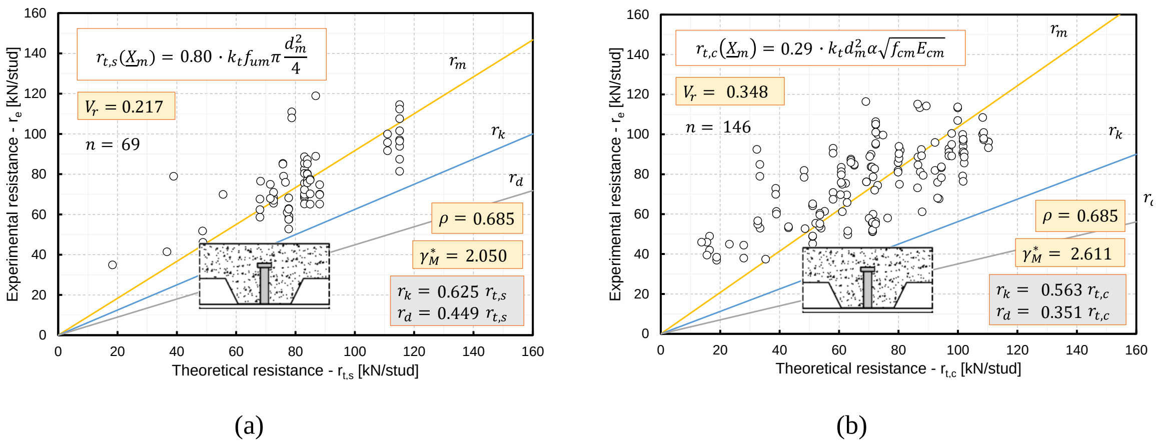
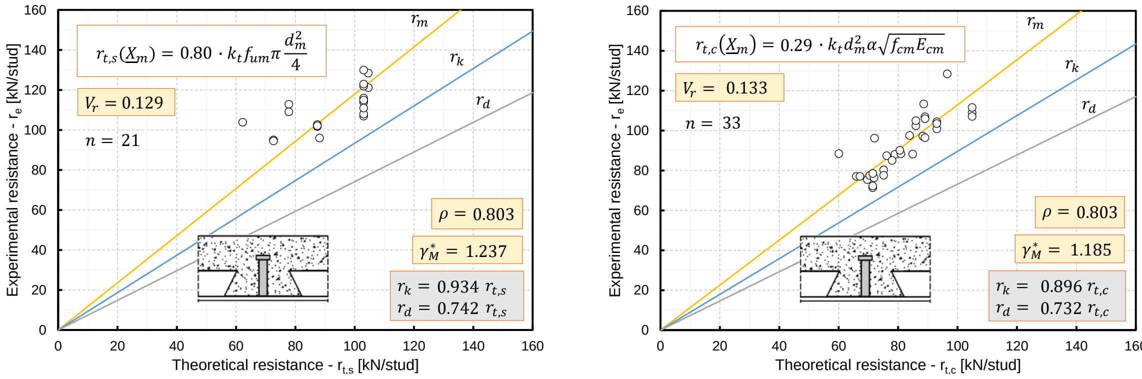


Fig. 8 Comparison between experimental and theoretical resistance according to EN 1994-1-1 design model for studs in open trough profiled steel sheeting (n=215 tests): (a) Eq.(2) for steel and (b) Eq.(3) for concrete failure.

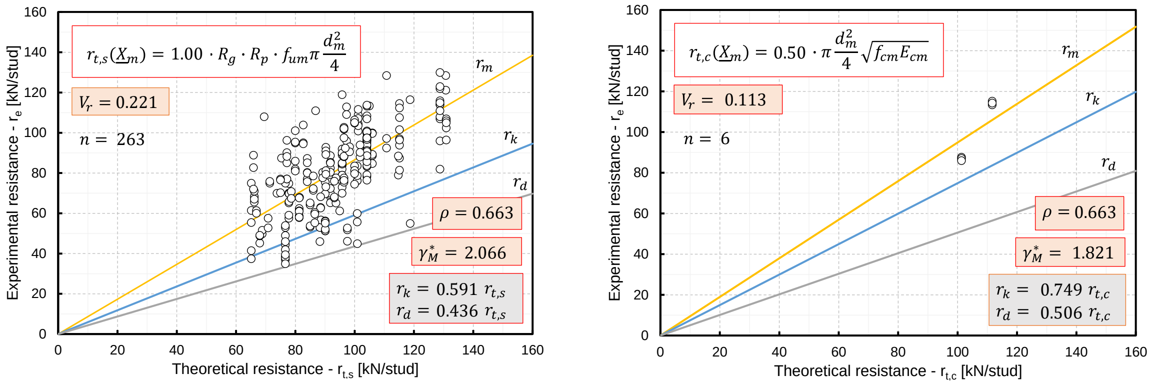


(a) (b)

Fig. 9 Comparison between experimental and theoretical resistance according to EN 1994-1-1 design model for studs in re-entrant profiled steel sheeting (n=54 tests): (a) Eq.(2) for steel and (b) Eq.(3) for concrete failure

5.2. Current AISC 360-16 design rules

Unlike the Eurocodes, the "design for strength using Load and Resistance Factor Design" (LRFD) given in AISC 360-16 (2016) provides for a global resistance factor ϕ_b=0.9 applied to the resulting flexural resistance of the composite beams. Although the design approach undertaken by the North American code does not explicitly provide an "equivalent" partial safety factor of the shear resistance of studs, the same statistical procedure was carried out in order to compare the statistical performance of different design models in a consistent manner. The plots in Fig. 10 show the comparison between experimental and theoretical resistance of studs according to the design model in AISC 360-16 (2016). As can be seen from these results, Eqs.(7) and (8) deliver a partial safety factor equal to 2.07 and 1.82 for steel and concrete failure, respectively. The reduction factors accounting for the use of the profiled steel sheeting apply only to the steel failure equation (Fig. 10(a)) that represents the governing case for 263 out of 269 representative push tests considered. As shown in Fig. 10(b), the reduced population size of the other 6 tests covered by the concrete failure equation justifies the low coefficient of variation V_r=0.113. From analyses of subsets of different profile geometry (Fig. 11 and Fig. 12), this design model shows better performance for predicting the stud resistance in re-entrant sheeting with a required partial safety factor $\gamma_M{}^*$=1.594 and a coefficient of variation V_r=0.224. However, the coefficient of correlation ρ is quite low.



(a) (b)

Fig. 10 Comparison between experimental and theoretical resistance according to AISC 360-16 design model (n=269 tests): (a) Eq.(7) for steel and (b) Eq.(8) for concrete failure.

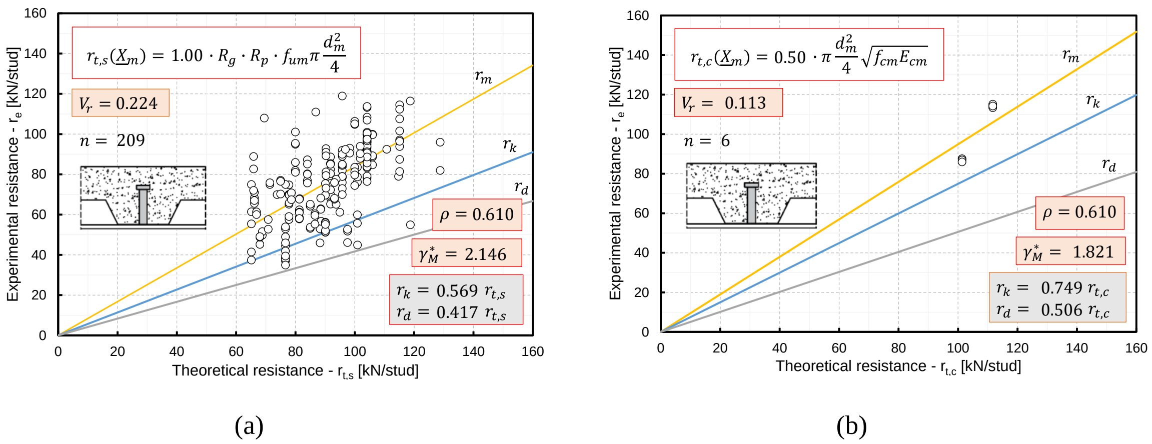


Fig. 11 Comparison between experimental and theoretical resistance according to AISC 360-16 design model for studs in open trough profiled steel sheeting (n=215 tests): (a) Eq.(7) for steel and (b) Eq.(8) for concrete failure.

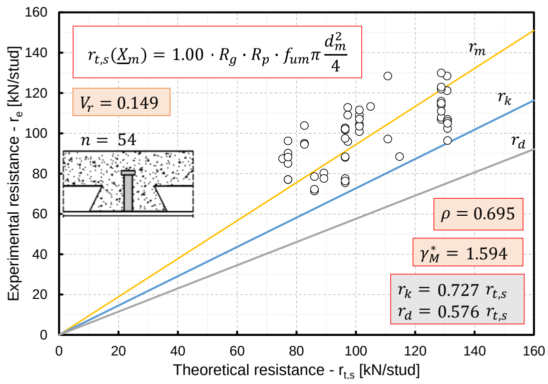
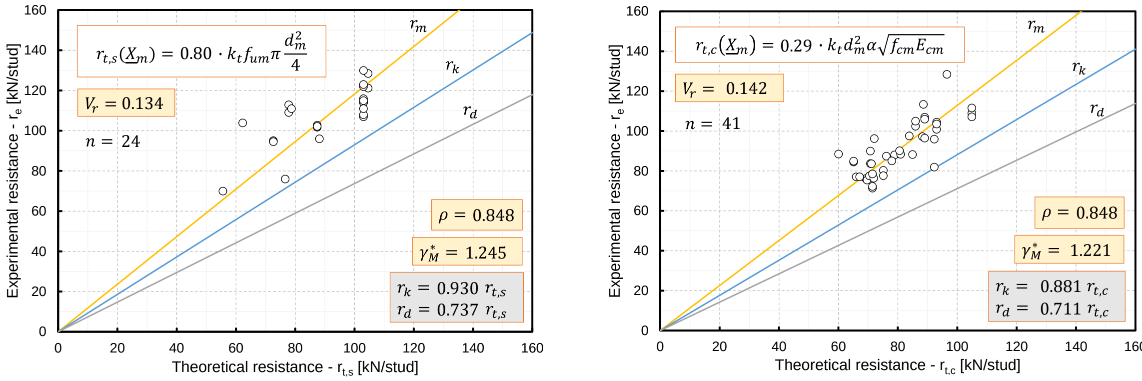


Fig. 12 Comparison between experimental and theoretical resistance according to AISC 360-16 design model for studs in re-entrant profiled steel sheeting (n=54 tests).

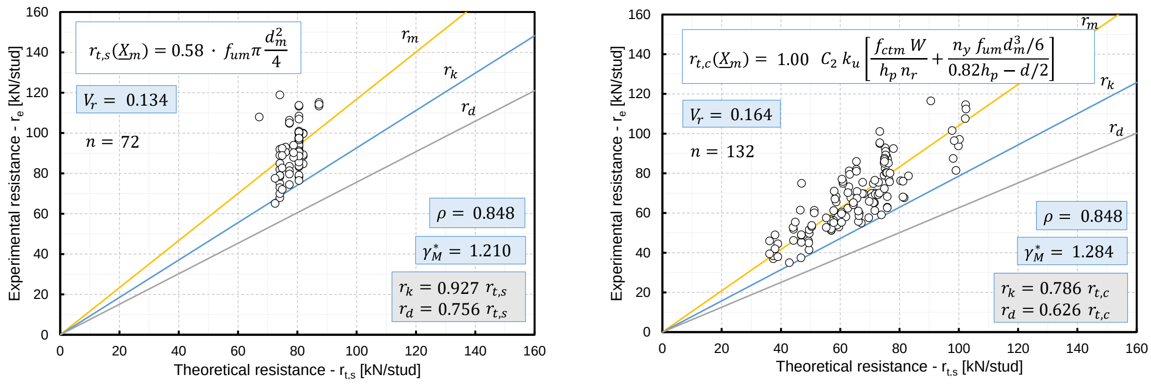
5.3. Luxembourg design model

As discussed in Section 2, the design model consists of an hybrid solution where the shear resistance of studs in profiled sheeting may be still calculated according to current EN 1994-1-1 (i.e. Eqs.(2) and (3)), only if specific geometrical conditions are satisfied. Conversely, if the configurations fall outside these boundaries, the new design equations given by Eqs.(13) and (14) should apply. Therefore, to perform the reliability analyses of the design models, the database was split into two different sub-databases: 65 tests lie in the field of applicability of current EN 1994-1-1 equations, while the remaining 204 tests belong to the domain of the new design equations. The individual results of the reliability analyses are presented graphically in Fig. 13 and Fig. 14 for the equations of EN 1994-1-1 and new equations, respectively. The linear correlation factor $\rho = 0.848$ was calculated from considering all test results (i.e. n = 269), which indicates that the design models seems to be suitable for this application. As can be seen from these plots, comparable results are obtained in all cases with a partial safety factor varying from 1.22 and 1.28. The coefficient of variation V_r for steel failure is approximately equal to 0.134 for both cases, i.e. Eqs.(13) and (14), and increases to 0.164 for the combined "stud in bending-concrete cone" failure expressions given by Eq.(14) and shown in Fig. 14(b). However, it is worth noting that the new equations cover most of the push tests (n=131) where slender geometries of sheeting are employed, but are not safely predicted by both the current European and North American codes.



(a) (b)

Fig. 13 Comparison between experimental and theoretical resistance according to "Luxembourg" design model in the field of applicability of current EN 1994-1-1 equations (n=65 tests): (a) Eq.(2) for steel and (b) Eq.(3) for concrete failure.

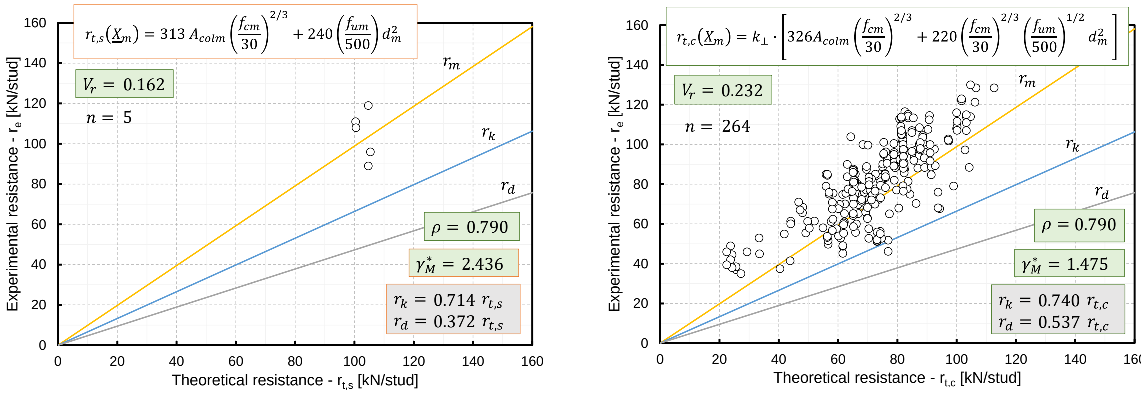


(a) (b)

Fig. 14 Comparison between experimental and theoretical resistance according to "Luxembourg" design model in the field of applicability of new equations (n=204 tests): (a) Eq.(13) for steel and (b) Eq.(14) for "stud in bending-concrete cone" failure.

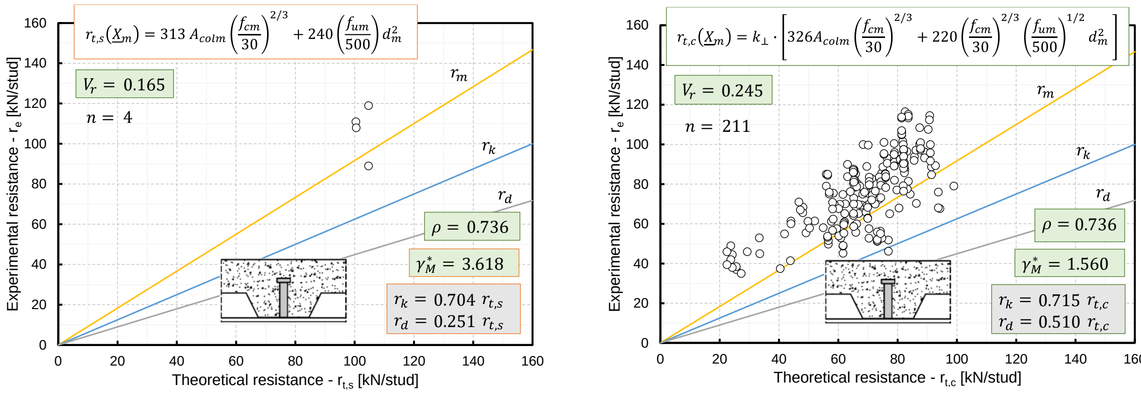
5.4. Stuttgart design model

From the reliability analyses of the "Stuttgart" design model, except for five push-out test, Eq.(21) is decisive and the corresponding results of the statistical evaluation are presented in Fig. 15. The linear correlation factor ρ=0.790 confirms the suitability of the design model for predicting the shear resistance of studs in profiled sheeting. The coefficient of variation V_r for Eq. (21) is 0.232 resulting in a required partial safety factor of 1.475. From the evaluation of different steel sheeting shapes given Fig. 16 and Fig. 17, it appears that the results improve significantly for re-entrant sheeting (see Fig. 17(b)). Conversely, the configurations using sheeting with open trough profiles are the most critical where the scatter between experimental and theoretical resistance values becomes more important. The coefficient of variation V_r of Eq. (21) increases from 0.169 to 0.245, from re-entrant to open trough profiles, respectively.



(a) (b)

Fig. 15 Comparison between experimental and theoretical resistance according to “Stuttgart” design model (n=269 tests): (a) Eq.(20) for steel and (b) Eq. (21) for concrete failure.



(a) (b)

Fig. 16 Comparison between experimental and theoretical resistance according to “Stuttgart” design model for studs in open trough sheeting (n=215 tests): (a) Eq.(20) for steel and (b) Eq. (21) for concrete failure.

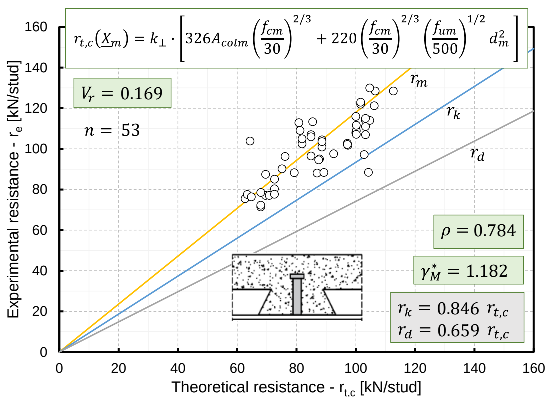


Fig. 17 Comparison between experimental and theoretical resistance according to “Stuttgart” design model for studs in re-entrant sheeting (n=53 tests).

6. Discussion of the results

The results of the statistical evaluations of the different design models presented are evaluated in this section. An important indicator of the quality of the design model is represented by the coefficient of variation V_r which consider not only the error terms (i.e. comparison with test results) but also the inherent variability due to the uncertainties of the basic variables. The values of V_r are summarized in the bar chart presented in Fig. 18.

Design equations currently given in European and North American codes reach a coefficient of variation between 22% and 32%. The application of the novel empirical “Stuttgart” design model provides a modest reduction to 16% and 23%. Smaller values for the coefficient of variation are delivered by the “Luxembourg” design model where V_r ranges between 13% and 16%. The good statistical performance of this model is related to the mechanical nature of Eq.(14) which covers almost half of the cases considered (132 out of 269 tests). Unlike alternative empirical equations, this expression is able to consider the actual failure modes occurring in the connection (Vigneri *et al.* 2019b) for open trough sheeting with narrower or deeper ribs.

Similar trends can be found in the values of the corrected partial safety factors γ_M^*, as can be seen in Fig. 19. The target value of the partial safety factor γ_V recommended by EN 1994-1-1 is equal to 1.25. Although no partial safety factor is defined at the level of the shear resistance of studs in the North American code, the design model of AISC 360-16 is also included in this evaluation for comparison purposes. A value of γ_V =1.25 is justified for the respective design resistance P_{Rd} if γ_M^* does not exceed the target value of 1.25. Conversely, γ_M^* >1.25 indicates that the use of γ_V =1.25 results in a design value that does not fulfil the safety requirements of EN 1990 (2002), i.e. the probability $P(r \leq P_{Rd})$ is higher than 0.1%. The results obtained from the reliability analyses show that current EN 1994-1-1 and AISC 360-16 deliver comparable values for the corrected partial safety factor ranging between 1.75 and 2.25. On the other hand, both newly proposed design approaches give more promising results with a partial safety factor below 1.5. Whilst Eq.(21) of “Stuttgart” model delivers γ_M^*=1.475, the resistance functions defined within the “Luxembourg” model reach values of γ_M^* between 1.21 and 1.284, which justifies the application of γ_V=1.25. It should be mentioned that the reduced population size when Eq. (8) and Eq.(20) apply, strongly affects the results which may not be representative of their real performance.

For the sake of completeness, the corresponding theoretical probability of not exceeding the design resistance $P(r \leq P_{Rd})$ was also determined for all resistance equations considered and included in Table 7. This confirms the suitability of the partial safety factor γ_V=1.25 for the Luxembourg model where the probability of non-exceedance of the design resistance ranges between 0.1% and 0.2%. The probability increases up to ca. 1% for Stuttgart model (only Eq.(21)) while current EN 1994-1-1 rules deliver a maximum value of 14.6%.

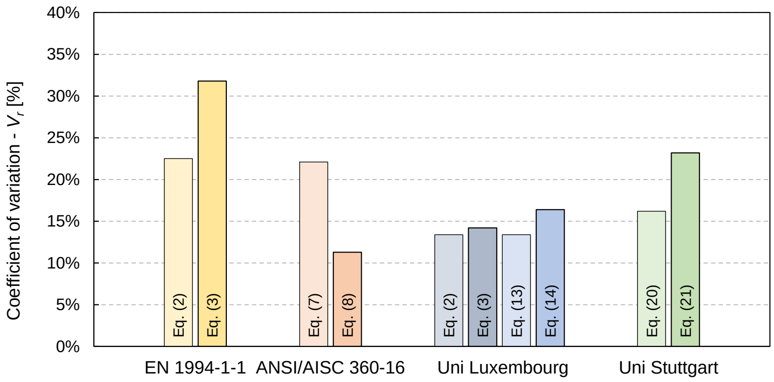


Fig. 18 Coefficient of variation V_r for different design models.

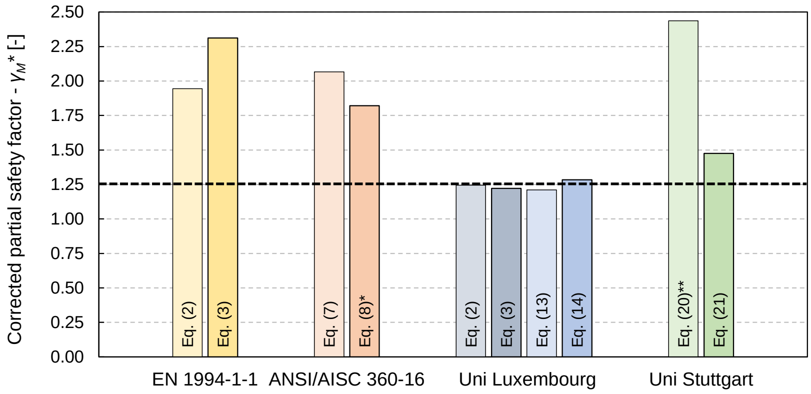


Fig. 19 Corrected partial safety factor γ_M^* for different design models. (The database contains *6 tests for Eq.(8) and **5 tests for Eq.(20)).

Table 7 – Main statistical parameters obtained from the reliability analyses of the design models.

	EN 1994-1-1		AISC 360-16		University of Luxembourg				Uni. of Stuttgart	
	Eq.(2)	Eq.(3)	Eq.(7)	Eq.(8)	Eq.(2)	Eq.(3)	Eq.(13)	Eq.(14)	Eq.(20)	Eq.(21)
n	90	179	263	6	24	41	72	132	5	264
ρ	0.701		0.663		0.848				0.790	
b	0.988	1.057	0.866	0.949	1.182	1.129	1.168	1.040	1.013	1.106
V_r	0.225	0.318	0.221	0.113	0.134	0.142	0.134	0.164	0.162	0.232
γ_M^*	1.944	2.312	2.066†	1.821†‡	1.245	1.221	1.210	1.284	2.436‡	1.475
$P(r \leq P_{Rd})$	14.6%	14.5%	23.0%†	61.8%†‡	0.1%	0.1%	0.1%	0.2%	86.6%‡	1.0%

† No partial safety factor is defined at the level of the resistance of studs AISC 360-16. These values are calculated for the sake of comparison.
‡ The results obtained may not be representative of the actual statistical performance of the resistance function due to the limited sample size.

7. Conclusions

Recent tests on studs within the ribs of modern trapezoidal, or 'open-trough', profiled steel sheeting have suggested that the predictions given by the current EN 1994-1-1 and North American specification (AISC 360-16) can lead to overoptimistic design resistances. This was recognised by the Eurocode 4 committee, who assigned Project Team CEN/TC250/SC4.T3 the responsibility to recommend an alternative design model. The two alternative design models that were considered were developed by the University of Luxembourg and the University of Stuttgart. The former consists of a combination of current EN 1994-1-1 rules and new mechanical-based equations, whereas the latter is based on empirically derived equations.

In the current work, reliability analyses were undertaken to investigate the performance of the four different design models for both open-trough and re-entrant profiled sheeting. From an initial database of 611 push-out tests, 269 cases were considered for the reliability analysis which, as well as ensuring that the results were valid over a wide range of geometrical and material properties, was significantly larger than the database used in the original calibration of EN 1994-1-1 (where only 57 tests were considered). It was found that a target partial safety factor of 1.944 and 2.312 was achieved by EN 1994-1-1 for the steel- and concrete-failure equations, respectively. Therefore, the recommended partial safety factor of γ_V =1.25 is not justified for the current design equations of EN 1994-1-1 for all the configurations considered. However, the current EN 1994-1-1 rules appear to be suitable for predicting the resistance of headed studs placed in re-entrant profiled sheeting as the target partial safety factor γ_M^*

is equal to 1.185. The analysis of AISC 360-16 equations leads to similar results with target partial factors of 2.066 and 1.821. However, since AISC provides a single global safety factor to be applied to the bending resistance of the composite beam rather than partial safety factors, these results do not necessarily imply the unsuitability of the equations given in the North American code. Further studies are recommended to assess the performance of this design model by determining the factors required for the bending resistance of the whole composite beam. Compared to EN 1994-1-1, the two alternative design approaches namely, the “Luxembourg” and “Stuttgart” model, show better statistical performance in terms of correlation with test results as well as safety. The coefficient of variation reduces to 0.162 and 0.232 for “Stuttgart” model while the hybrid “Luxembourg” model exhibits a value of V_r ranging between 0.134 and 0.164. Whilst the calculation of the corrected partial safety factor indicates that the “Stuttgart” model would require a partial safety factor of approximately 1.47, it appears that the “Luxembourg” model permits the use of γ_V =1.25. Furthermore, the mechanical nature of these equations allows for the consideration of the actual failure modes observed in the push-out tests. Given the fact that the Luxembourg design model is the only model that achieved the target values required by EN 1990, it is recommended as a potential candidate for inclusion within the second generation of Eurocodes.

Acknowledgements

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