Full paper

Carrying capacity of back-to-back cold-formed thin-walled U profiles subjected to compression

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Abstract

Thin-walled cold-formed members are mainly used in rack structures due to their light weight and the variety of cross-sections of different shapes that can optimize the design process. However, in cases where bigger span distances or higher resistances are required, the use of single members is not adequate and built-up sections deem to be necessary. To form such sections, single sections are connected with intermediate fasteners, with or without packing plates. Although the effect of these intermediate connections is considered in some international standards such as the American and the Australian ones, the latter is totally neglected in EN 1993-1-3. Rules are only found in EN 1993-1-1 related to closely-spaced battened members, but their possible extension to cold-formed thin-walled sections could be questionable and needs further validations. This paper presents numerical and analytical studies aiming at investigating the validity and applicability of these design rules for predicting the buckling resistance under pure compression of a battened built-up member consisting of two back-to-back thin-walled U profiles. In this framework, a specific attention is paid to the effects of the shear stiffness associated to the batten plates. Finally, design recommendations are provided.

Keywords

Buckling, Thin-walled members, Built-up section, Stability, Closely spaced member

1 Introduction

Thin-walled cold-formed built-up members are often used in industrial buildings due to their advantages, such as the strength-to-weight ratio, the ease on-site assemblage and their stability behaviour. To form such sections, single profiles are welded or connected with intermediate fasteners, screws or bolts with or without battened plates, resulting in different types of sections such as open, closed or battened ones. Built-up section members prevent the buckling of the individual components and as the section is symmetrical, at least according to one of its axes, the eccentricities between shear and gravity centres are eliminated, leading thus to higher member stability and reduced global slenderness [1]. In rack structures, battened built-up members are usually selected for diagonals used as bracing members, due to space limitations at the connection level and the higher carrying capacity demands. Being so, they are subjected either to compression or tension loads. However, as the latter loading case is rarely critical for the design of these members, emphasis is given here to their design under compression.

Besides built-up members are commonly used in industrial buildings, there is still a lack of codified design approaches [2]. More precisely, although the effect of the intermediate connections is considered in some international standards such as the American [3] and the Australian ones [4], this effect is totally neglected in EN 1993-1-3 [5], where only recommendations for the selection of the buckling curves of a closed built-up thin-walled cold-formed cross-section are given. Rules in Eurocodes related to closely-spaced battened members are only found in EN 1993-1-1 [6], where limitations of the maximum distances between the intermediate connections along the members are given. In cases where these limits are not satisfied, the shear stiffness of the connection should be taken into account via the critical load calculation according to both EN 1993-1-1 and CEN/TR 1993-1-103 [7]. However, this approach is recommended for hot-rolled profiles and its possible extension to cold-formed thin-walled sections could be questionable and needs further validation.

Extensive research has been carried out regarding the structural behavior and the design of built-up thin-walled cold-formed members made of two back-to-back channel or U sections. Meza et al. [8] experimentally investigates the interaction between the individual components under increasing loading and the effect of the connector spacing on the cross-sectional moment capacity. Anbarasu et al. [9] focuses on the axial capacity and nonlinear deformation response of pin-ended columns under monotonic axial loading. Mahar et al. [10] proposes a design procedure for the local buckling strength prediction of cold-formed steel built-up columns based on the Effective Width Method. Whittle et al. [11] compares the experimental resistances of a huge number of tests on built-up members formed of channels to their theoretical buckling capacities and proposes design recommendations. However, in all these research, the two component channel profiles are welded, connected with spacers or with fasteners but without using a battened plate. Only Minghini et al. [12] deals with built-up columns composed of closely spaced channel sections, but in this case the profiles are connected with FRP battens and not with classical steel plates.

This paper presents investigations conducted on the validity and applicability of the existing design rules provided in EN 1993-1-1 for determining the buckling resistance of a built-up member consisting of two back-to-back thin-walled U profiles under pure compression, accounting for the effects of the shear stiffness associated to the batten plates. To achieve this, a number of parametrical numerical studies on these type of members have been conducted with ABAQUS software [13]. The numerical results have been then compared with the analytical ones given in EN 1993-1-1. It should be also mentioned that there are no substantial differences between EN 1993-1-1 and its new version prEN 1993-1-1 [14] regarding the design of these members. Finally, design recommendations are provided.

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2 Provisions of EN1993-1-1

According to EN 1993-1-1, clause 6.4.4(1), built-up members connected through battened plates, should be checked for buckling as a single integral member and ignoring the effect of shear stiffness when the maximum spacing centre-to-centre distance between the interconnections “d_{ch}” (see Figure 1) is smaller or equal to $15i_{min}$, where i_{min} is the minimum radius of gyration of one chord. If this condition is not met, the effect of shear stiffness should be then taken into account. The buckling design resistance of the built-up member may be determined from:

$$N_{b,Rd} = \begin{cases} \frac{\chi_{min} A f_{yb}}{\gamma_{M1}} & if\ A_{eff} = A \\ \frac{\chi_{min} A_{eff} f_{yb}}{\gamma_{M1}} & if\ A_{eff} < A \end{cases} \quad (1)$$

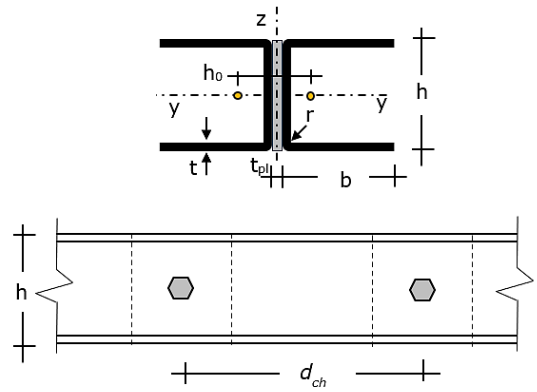

Figure 1 Definition of the built-up cross section axes and the distance between the battened plates

where the buckling reduction factor χ_{min} is determined as a function of the relevant relative slenderness $\bar{\lambda}$ of the compression member using buckling curve *a* or *b* for buckling about y-y or z-z axes, respectively (see **Figure 1** for the definition of the axes), in combination with the basic yield stress f_{yb}. The effective area A_{eff} is calculated according to the provisions of EN 1993-1-3, accounting for local buckling phenomena; there are no modifications regarding the calculation process of the effective properties in the new version of EN 1993-1-3, namely prEN 1993-1-3 [19].

For members in which the maximum distance between the interconnections is $d_{ch} \leq 15i_{min}$, the critical axial force N_{cr} is given by Euler’s theory based on the gross cross-sectional properties. For members in which the maximum distance d_{ch} is higher than $15i_{min}$, the critical axial force considering the influence of the shear stiffness is calculated according to [7] as follows:

$$N_{cr,Sv} = \frac{1}{\frac{1}{N_{cr}} + \frac{1}{S_v}} \quad (2)$$

where N_{cr} is the flexural critical axial force of the built-up member according to Euler’s theory considered as integral and by neglecting the influence of the shear stiffness; the latter is accounted via the term S_v and is determined according to EN 1993-1-1 as follows:

$$S_v = \frac{1}{\frac{d_{ch}^2}{24EI_{ch}} + \frac{d_{ch}h_0}{12EI_{pp}}} \leq \frac{1}{\frac{d_{ch}^2}{2\pi^2 EI_{ch}}} \quad (3)$$

where the moment of inertia of the effective part of the battened plate I_{pp} is given by equation (4), in which *B* is the inside diameter of the bolt head, d_0 is the diameter of the hole and the thicknesses are defined in Figure 1.

$$I_{pp} = \frac{\pi\left(B+2t+t_{pl}\right)^4 - \pi d_0^4}{64} \quad (4)$$

It can be seen through equation (3) that the shear stiffness depends on the flexural stiffness of the batten and the flexural stiffness of the column. In cases where the batten and the chords are also subjected to direct shear forces, an extension of this equation is proposed in CEN/TR EN 1993-1-103, but it is not considered here.

In order to check the validity and the applicability of this design rules to built-up thin-walled cold-formed sections, a numerical parametrical study is performed, divided into two parts. The first part studies members with interconnection distances $d_{ch} \leq 15i_{min}$, in order to check the validity (i) of the code provisions (i.e. integral member response) and then (ii) of the buckling resistance predictions. In the second part, members with interconnection distances $d_{ch} > 15i_{min}$ are investigated in order to check the influence of the distance of the battens on the buckling resistance of the member. These numerically obtained results are checked against the code provisions explained in this section.

3 Details of the studied members

The details of the parametric study are addressed and discussed in the following. It should be mentioned that only bolted connections with clearance are investigated in this paper. Dimensions representative from practical cases have been adopted for the geometrical properties (sections, holes, etc).

3.1 Cross-sections

For the parametrical study, 6 built-up sections have been selected; their geometrical properties are reported in Table 1. Moreover, the diameter of the holes on the U profiles is 12,5 mm while the used bolts are for all cases M12 (8.8) partially preloaded.

Table 1 Geometrical properties of the built-up sections

No	Profile	$I_{y,2U}$ [mm⁴]	$I_{z,2U}$ [mm⁴]	$A_{2U,eff}$ [mm²]	h_0 [mm]
1	2U60/60/2,0	448685	702036	439,6	50,1
2	2U60/60/3,5	717709	1182649	982,2	50,3
3	2U70/70/2,0	724605	1083773	460,8	56,8
4	2U70/70/4,0	1308165	2074343	1297,3	57,0
5	2U80/80/3,0	1571865	2330341	923,2	63,6
6	2U80/80/4,0	2003925	3047405	1395,8	63,6

Hereafter, the built-up section will be mentioned with the index "*2U*", while the single U profile with the index "*ch*" (chord). The moment of inertia about y-y axis is twice the relevant value of the single U profile, while about z-z axis is given by the formulae $I_{z,2U} = 2I_{z,ch} + 0{,}5A_{ch}h_0^2$. It should be said that all the calculations are based on the middle-line dimensions concept given in §7.3.1 of prEN 1993-1-3 and by neglecting the rounded corners, as the two criteria expressed in §7.3.1.(3) are fulfilled for all the profiles considered in this study. Although the interpretation of §7.3.1.(3) is quite ambiguous regarding the cases where the influence of the rounded corners should be accounted for in the calculations, it has been demonstrated by Dubina et al. [20] that the influence of the rounded corners to the inertia properties is ranging from 1 to 5% for profiles with $r = t$ and thicknesses t less than 5,0 mm. Given that, for all the considered profiles, the radius r is always equal to the thickness t, and the maximum considered thickness is 4,0 mm, it can be concluded that this influence remains rather small (less than 4%).

3.2 Battened plated

The thickness of the battened plates is assumed to be constant for all the analyses and equal to 10 mm; this value corresponds to ratios $t_{plate}/t_{section}$ ranging from 0,2-0,5, covering thus a reasonable range of practical cases. In addition, a square shaped with a width equal to the height of the connected U profiles is selected, while the diameter of the holes is assumed to be equal to 13,5 mm. Moreover, two different interconnection distances have been investigated, $d_{ch} = 15i_{min}$, and $d_{ch,1} = 30i_{min}$.

3.3 Steel grades

The steel grade of the U profiles is HX420LAD (f_{yb} = 400 N/mm² according to prEN 1993-1-3), for the battens is S355 (f_y = 355 N/mm²), while for the M12 (8.8) bolts f_{yb} is equal to 640 N/mm².

3.4 Preloading of the bolts

For all the studied members, preloaded M12 (8.8) bolts are used, with a tightening torque of 62 N·m. This value, which is a common value used in practical applications in rack structures, corresponds to about 68% of the minimum preloading force according to EN 1090-2 [21].

3.5 Member's slenderness

Members with different lengths have been studied in order to cover an extended range of global slenderness (i.e. from 0,34 to 1,8).

4 Description of the numerical model

The numerical investigations were carried out with ABAQUS® non-linear finite element software. The numerical tool was first validated through comparisons with existing experimental test results, where battened back-to-back angle profiles have been tested under compression loads [22]; the validation process is omitted here due to length limitations of the paper.

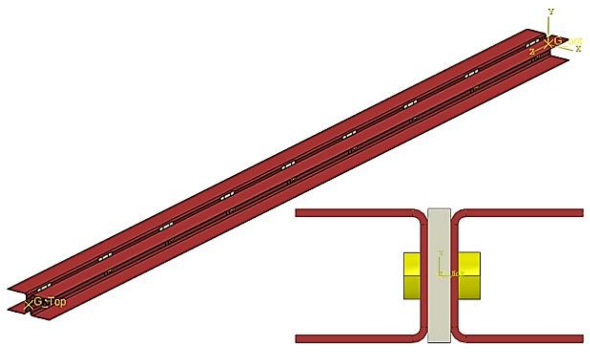

Figure 2 FEM model and vertical cut at the position of the bolt

An overview of the 3D model of the built-up member considered for the parametrical study is shown in Figure 2. Isoparametric 4-node shell elements with reduced integration (i.e. type S4R) have been used for the U profiles and the steel plates, while solid 8-node linear brick C3D8R elements were selected for the bolts. The meshing density is increased in the region of the connections in order to enable the numerical simulation to correctly capture the contact behaviour. In addition, all the samples used for the numerical investigations have been modelled as pinned at their extremities, where specific constraints distribute uniformly the external applied loads and avoid thus any local failure at the load application point.

In all simulations, the following two steps are applied. First, a linear buckling analysis (LBA) is performed in order to obtain the elastic critical loads as well as the associated eigenmode shapes. These eigenmode shapes are then used as initial imperfection shapes within the second simulation step, through which a full non-linear analysis (GMNIA) is carried out in order to obtain the maximum load carrying capacity of the built-up member. In all simulations, the applied compressive load is increased up to failure. The shape and the amplitude of the initial imperfections implemented in the GMNIA are explained in the following. For slender members ($\bar{\lambda} > 1{,}2$), for which the relevant buckling mode obtained through the LBA is a pure flexural one, an equivalent bow imperfection with an amplitude equal to $L/500$ (L being the length of the built-up member) and a shape affine to this first global elastic instability mode is selected, according to prEN 1993-1-1, clause 7.3.3.1(4); for these cases, minor axis buckling is predominant. For short to intermediate slenderness members ($0{,}2 \leq \bar{\lambda} \leq 1{,}2$), for which the relevant buckling mode obtained through the LBA is a local buckling, the two imperfections detailed below have been checked separately and the one that gives the smallest resistance is finally adopted:

a. an equivalent local imperfection equal to $b/125$ (b being the width of the outstand element of the profile) with shape affine to the first local elastic instability mode, in line with the recommendations of prEN 1993-1-14, Table 5.6-5.7 [23];

b. an equivalent global-local combined imperfection with magnitude $L/1000$ (L being the length of the built-up member) and shape affine to the first global-local combined mode, according to prEN 1993-1-14, clause 5.5(4) for combined eigenmodes of cold-formed profiles.

The adoption of these imperfections is quite reasonable and in line with the physical expectations, as local imperfections are relevant for rather short members, combined local-global imperfections are relevant for short to intermediate ones, while global imperfections are mainly affecting the more slender members.

For all steel grades, a simplified bi-linear elastoplastic material behaviour law with strain hardening has been adopted for the GMNIA's [23]. The Young modulus is E = 210000 N/mm^2 for all steel grades. Furthermore, residual stresses are not implemented separately in the numerical model, but are taken into account through the above mentioned equivalent imperfections.

In order to represent the real stiffness of the built-up section, several contact regions were defined. For the tangent contact between the nut and the U profile and the contact between the battened plates and the U profile, a friction coefficient equal to 0,2 has been applied; this value is in line with Table 5.13 of prEN 1993-1-8 [24] and considered as a lower bound. Nonetheless, as the bolts are not fully preloaded as explained above, the value of the friction coefficient has been shown to have very low influence [22]. For the normal contact behaviour, a "hard" contact law which allows separation of the surfaces in contact has been applied. Moreover, the possible sliding before contact can be accounted for through the actual clearances of the bolt holes that have been modelled. A tightening torque of 62 N·m is applied to all the bolts.

5 Results and discussion

The numerical results of the parametric study, as well as their analytically obtained predictions, are presented and discussed in the following. Table 2 summarizes the geometry of the studied members (cross-section and member's length L); the term $m|d_{ch}$ indicates the number of the packing plates m per member and their centre-to-centre interval d_{ch} for the two considered interconnection distances (i.e. d_{ch} = 15i_{min}, and $d_{ch,1}$ = 30i_{min}).

Table 2 Geometry of the studied members

No	Profile	L [mm]	d_{ch}=15i_m	$d_{ch,1}$=30i_m
			$m|d_{ch}$ [-|mm]	$m|d_{ch}$ [-|mm]
1	2U 60/60/2,0	800	3|280	2|570
2	2U 60/60/2,0	4000	15|280	7|570
3	2U 60/60/3,5	2300	9|270	4|550
4	2U 60/60/3,5	3500	13|270	7|550
5	2U 70/70/2,0	1400	5|330	2|670
6	2U 70/70/2,0	2100	7|330	3|670
7	2U 70/70/4,0	3000	10|310	5|640
8	2U 70/70/4,0	3700	12|310	6|640
9	2U 80/80/3,0	2500	7|400	4|750
10	2U 80/80/3,0	3000	9|360	4|750
11	2U 80/80/4,0	3200	9|365	5|740
12	2U 80/80/4,0	3800	11|365	5|740

It should be mentioned that comparisons between analytical and numerical results were first made at the level of the critical loads and then on the ultimate buckling resistance of the members.

5.1 Critical buckling loads

For centrally loaded compressed members with doubly symmetric sections, as the built-up 2U profile, two global buckling modes (flexural, torsional) may occur; their critical loads can be analytically determined according to the technical report CEN/TR 1993-1-103 and the well-known stability theory. Although local buckling has to be accounted for thin-walled elements, it is not explicitly considered in the design procedure proposed in the Eurocodes but is accounted for through the effective properties of the cross-sections.

For these parametric investigations, the members reported in Table 2 have been considered and their global buckling critical loads have been calculated analytically and numerically. The numerical critical loads are obtained through a linear buckling analysis (LBA) and always correspond to the first global buckling mode that is a flexural one; torsional modes are not relevant at least for the studied members. The analytical critical loads are calculated according to Euler's formulae for interconnection distances $d_{ch} \leq 15i_{min}$, and according to eq.(2) for distances $d_{ch} \leq 30i_{min}$. Due to lack of space, the tables with the obtained values are omitted and only the main conclusions related to this study are presented in the following.

For members with interconnection distances $d_{ch} \leq 15i_{min}$, both the analytical calculation and the LBA provide almost identical results with a mean value of the ratio $N_{cr,n}/N_{cr,an}$ equal to 1,00 and a COV of 0,6%. Moreover, it has been observed that all the considered specimens behave as integral members, validating thus the code provision saying that the effects of shear stiffness can be neglected as far as $d_{ch} \leq 15i_{min}$.

For members with interconnection distances $15i_{min} \leq d_{ch} \leq 30i_{min}$, the flexural critical load accounts also for the effects of the shear stiffness of the plates; both the analytical calculation and the LBA provide quite similar results with a mean value of the ratio $N_{cr,n}/N_{cr,an}$ equal to 1,05 and CoV of 1,56%. It can be therefore concluded that the procedure described in EN 1993-1-1 for the determination of the axial buckling critical load of a built-up member where the effect of shear stiffness is accounted for, provides quite accurate and safe results, without being too conservative.

5.2 Buckling resistance

Figure 3 presents the ratio of the numerically over the analytically determined characteristic buckling resistances of the considered built-up members (see Table 2 for geometry details). The analytical characteristic buckling resistances under pure compression ($N_{b,Rk}$) are calculated according to EN 1993-1-3. As different buckling curves are proposed for buckling about y-y and z-z axes (see Figure 1 for the definition of the axes), both resistances are calculated as explained in §2 and the minimum one finally governs the design; for all cases, buckling around y-y axis is predominant. The numerical ultimate load ($N_{ult,n}$) corresponds to the maximum load obtained through the GMNIA analyses.

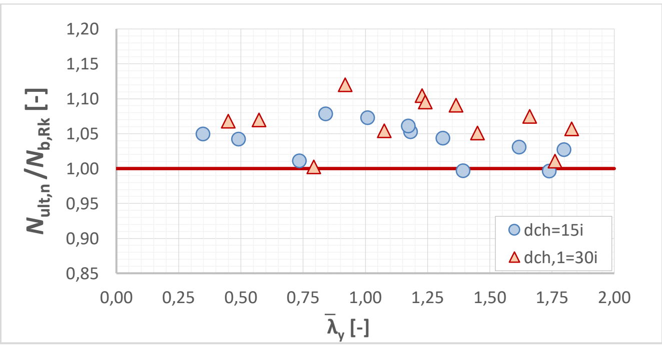

Figure 3 Comparison of the numerically ($N_{ult,n}$) and analytically ($N_{b,Rk}$) obtained resistances results with the buckling curves of EN 1993-1-1

For all specimens, the failure modes obtained numerically are in line with the applied initial imperfections, confirming thus their selection. Furthermore, it can be easily seen that there is a very good agreement between the numerical resistances and the ones calculated according to Eurocode provisions, with the latter being always on the safe side; the mean value of the ratio $n = N_{ult,n}/N_{b,Rk}$ is equal to 1,04 (CoV of 2,6%) and 1,07 (CoV of 3,4%) for plate distances of $d_{ch} = 15i_{min}$ and $d_{ch,1} = 30i_{min}$ respectively. Furthermore, it has been again seen that all the considered specimens with interconnection distances $d_{ch} \leq 15i_{min}$ behave as integral members, validating thus the code provision.

Figure 4 shows the comparison of the numerical resistances obtained for plate distances equal to $30i_{min}$ to the numerical resistance of the integral member (i.e. $d_{ch} \leq 15i_{min}$). It can be seen that, by doubling the distance of the battened plates, there is a reduction of the ultimate buckling resistance from 1 to 9% compared to the resistance of the integral member.

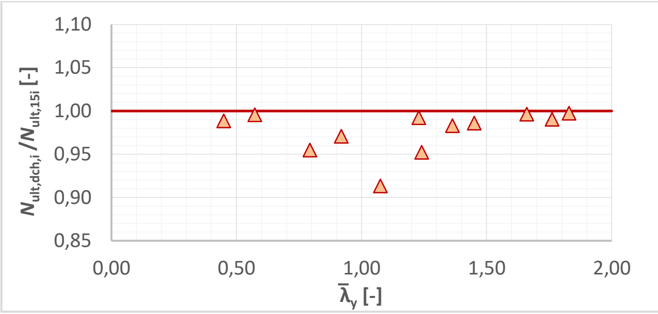

Figure 4 Reduction of the numerically determined resistance due to the increased distance between the battened plates

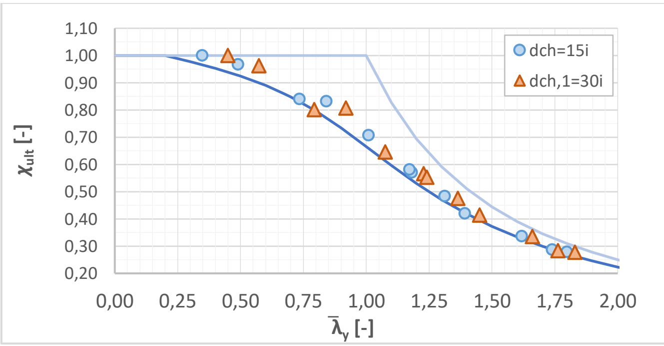

Figure 5 Comparison of the numerical results with the buckling curves of EN 1993-1-1

Figure 5 shows the numerically determined reduction factor $\chi_{num} = N_{ult,n}/(A_{eff} \cdot f_{yb}) \leq 1,0$ versus the global slenderness $\bar{\lambda}_y$ or $\bar{\lambda}_{y,Sv}$ for plate distances of $15i_{min}$ and $30i_{min}$ respectively; buckling curve *a* of EN 1993-1-1 as well as the Euler's buckling curve are illustrated too. It can be easily observed that all the results are above buckling curve *a*, validating thus the analytical procedure. Subsequently, the analytically determined buckling resistance of closely spaced built-up back-to-back sections made of two channel profiles provide safe and quite accurate results.

6 Conclusions

Through these studies involving numerical and analytical aspects, the following conclusions may be drawn:

– for battened plate distances smaller than $15i_{min}$, the cold formed built-up member behaves as a single integral member and its buckling resistance should be evaluated ignoring the effect of shear stiffness;

- for battened plate distances equal to $30i_{min}$, the effect of shear stiffness should be taken into account in the evaluation of the buckling resistance of the built-up member;
- a battened plate distance equal to $30i_{min}$ can reduce the buckling resistance of the member up to 9%, compared to the resistance of a member with plate distances of $15i_{min}$;
- the provisions of EN 1993-1-1 for closely spaced built-up members can also be safely applied for the determination of the buckling resistance of thin-walled cold-formed profiles, in combination with EN 1993-1-3 for the selection of the buckling curves and EN 1993-1-103 for the definition of the critical loads.

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