10th International Conference on

Steel and Aluminium Structures (ICSAS24)

Rio de Janeiro, Brazil, 5-7, June, 2024

BOLTED CONNECTIONS BETWEEN THIN-WALLED AND THICK ELEMENTS MADE OF COLD-FORMED SECTION

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Keywords: Bolted connection; Cold-formed section; Thin-walled member; Rack structure; Component method.

Abstract. *The market of self-supporting storage rack structures is extremely competitive requiring a constant evolution towards the development of bigger and higher projects for sake of economic optimisation. Up to now, such structures are mainly composed of cold-formed thin-walled profiles as primary elements so as to reduce the total weight. However, the need of bigger and taller structures that are nowadays appearing cause higher internal loads in the columns and so an increase of the column cross-section dimensions, which cannot be anymore considered as thin-walled, is required. This leads to uncommon connections where thin-walled cold-formed beams are connected to "thick" column profiles. The proposed paper presents studies conducted on bolted beam-to-column hinged connections between thick and thin-walled profiles. First, a critical review of European normative context and, in particular, of the component method as recommended design approach for joints is presented. Then, based on the available equations, design recommendations for the studied joint are proposed. Finally, an alternative connection is presented and design recommendations are given.*

1 INTRODUCTION

Due to the competitivity in the market of self-supporting storage rack structures, the optimisation of these solutions is a key issue and the use of elements made of cold-formed section allows to reach efficient solutions. In particular, the use of C or Z cold-formed sections for the primary elements provides lighter solution, allowing a faster erection on site and a more efficient transportation. On the other hand, the "self-supporting" character of these structures induces the need to sustain higher loads due to the external actions such as the wind and the snow. This results in thicker column cross-sections to provide more resistance while the use of thin-walled ones remains possible for the beams.

In addition, as the majority of self-supporting racks are braced structures, the beam-to-column connections are hinged for sake of simplicity and economy and, for practical reasons, almost all the connections are bolted on site. Therefore, bolted connections between thick and thin elements are met in such structures, and should be adequately designed, considering both resistance and ductility requirements.

EN1993-1-8 [1] is the European design code dealing with steel connections, where recommendations, based on the component method, for rigid and semi-rigid joints are provided. However, the latter is not covering explicitly the design of pinned joints. This joint typology will be covered in the next generation of this code where an annex specifically dedicated to the design of such joints will be reported (prEN1993-1-8 [2]). This annex has been developed on the basis of the design recommendations in terms of resistance and ductility criteria to be respected provided in [3]. For a steel joint connecting members made of cold-formed sections, EN1993-1-3 [4] has to be used. But, as detailed here below in §2.2, its field of application is rather limited as few recommendations are given compared to EN1993-1-8. In addition, EN15512 [5] is not providing clear equations to characterise a beam-to-column joint of a self-supporting storage rack, as this norm is dedicated to the design of storage rack structures with adjustable pallet racking systems, where fully perforated columns are used. It is for this reason that this study has been initiated.

Only few studies have been conducted about pinned connection between cold-formed members, such as [6] where the behaviour of a beam-to-column clip angle bolted connection subjected to shear force is experimentally investigated. In fact, most of the research [7]-[8] about connections made of cold-formed members involves joints where a gusset plate which is bolted to two back-to-back columns is used. However, the latter exhibit a semi-rigid behaviour and cannot be assumed as hinged joint.

The present paper gives an overview of the available provisions and design recommendations for the design of a bolted steel beam-to-column connection between a C-beam and a square hollow section, in the different normative relevant present and future codes, i.e. EN1993-1-3, EN1993-1-8, prEN1993-1-3 [9] and prEN1993-1-8; the differences between the current and the forthcoming versions are also highlighted. In order to check the applicability of the available recommendations and to derive a simple design method, a typical connection used in self-supporting rack structures that can be assimilated as pinned is considered. Finally, an alternative optimised joint solution is proposed.

2 CODE PROVISIONS

In the following sections, the design rules available in the relevant codes and their limitations are addressed and discussed.

2.1 Connections between thick members

As mentioned above, the reference code for the design of a steel connection is EN1993-1-8. Although no condition about the thickness of the connected members is given in the current version of the code, it is stated in its forthcoming version that the provisions apply for a plate thickness equal or greater than 3 mm.

The code gives first some preliminary propositions, such as the maximum and minimum spacing between edges and bolts, the bolt that can be used, the steel grade of the connected plates and the design resistance for individual fasteners subjected to shear in normal round holes. Then, it provides recommendations to apply the well-known component method, which is a suitable procedure to estimate the mechanical properties of joints that can be subjected to different type of loading situations. In particular, EN1993-1-8 provides equations to characterise the resistance and stiffness of different components as well as formulae to compute the resistance and rotational stiffness of lap joints and moment resisting joints made of elements with I or H section. This is a strong limitation for this research, as other types of sections are usually preferred in self-supporting rack structures. In addition, as stated above, the design of pinned joints is not covered in the present draft of EN1993-1-8. However, recommendations for such joint typology have been proposed in [3] and will be presented in the forthcoming version of this code. Indeed, Annex C of prEN1993-1-8 brings guidelines to design three distinct pinned connection configurations: (i) double angle web cleat, (ii) fin plate and (iii) partial end-plate connections. Again, their validity is limited to joints connecting I and H hot-rolled sections. These guidelines reported in Annex C include recommendations to estimate the design resistance and ductility criteria that should be respected so as to guarantee the “pinned” character of these joints.

2.2 Connections between thin members

When structures, and especially joints, are built with thin members made of cold-formed sections, the relevant code to be used is EN1993-1-3, where some recommendations are available for the design of bolted connections. This code is referring to EN1993-1-8 for the general design concepts and proposes specific rules to account for the “thin” character of the members when relevant. In EN1993-1-3, it is clearly stated that the provisions apply only to elements made of cold-formed sections with thickness lower or equal to 3 mm and higher or equal to 0.75 mm.

For bolted connections subjected to shear, the recommendations of Table 8.4 in EN1993-1-3 apply where equations to evaluate the bearing resistance, the net section resistance and the shear resistance of a bolt are given. Ductility conditions are also proposed. Some restrictions for applying those design formulae are expressed, especially about the distance between bolts and edges, where the minimum spacing values for thin sections differ from the ones provided for thicker sections. Finally, it has to be mentioned that the code does not provide any equation to assess the bearing resistance of an inner bolt row.

Within the present study, comparisons between EN1993-1-3 and prEN1993-1-3 have been made. The major difference is that the forthcoming version states that the above-mentioned Table 8.4 is applicable for material thickness between 4 mm and 0.75 mm, while the maximum thickness proposed in the existing version is 3 mm. Some other small differences about the equation notations have been noticed, especially for the ones about the bearing resistance and the net-section resistance.

The other normative document that could be used, when thin sections are considered, is the EN1993-1-5 [10], but no information is provided for what concerns the design of joints.

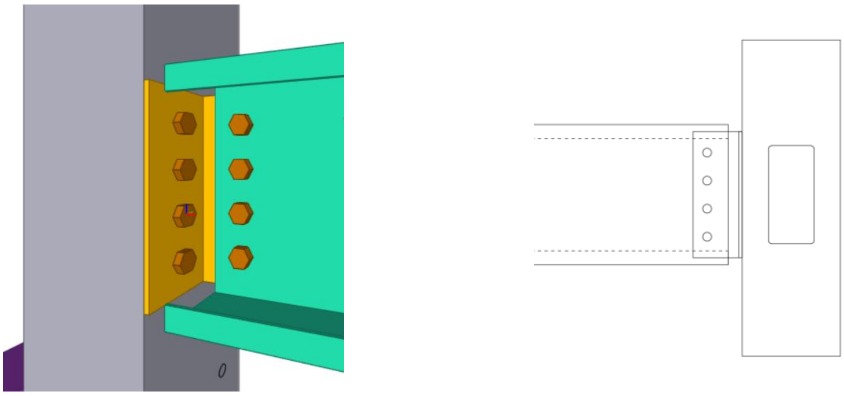
3 ANALYTICAL INVESTIGATIONS AND DESIGN METHOD

In this part, the connection that has been investigated is first presented. Then, existing recommendations for its design have been analysed and adapted when required to provide an adequate design method.

3.1 Presentation of the studied connection

In order to apply the derived design method to a practical case, a first simple pinned connection between a cold-formed thin-walled C-section beam and a cold-formed square hollow column is considered. The connection is made of an L-profile bolted to the web of the C-section and to the face of the square hollow-section. As the section of the column is closed, a “handhole” in the column face is realised to allow the tightening of the column face bolts. On each leg of the cleat, one bolt column is used, and the number of rows vary according to the height of the C-profile. The 3D configuration is schematized in Figure 1(a) while a longitudinal view of it is given in Figure 1(b).

This connection typology could be seen as a header plate one from the column side and a fin plate one from the beam side. Subsequently, the design recommendations provided for both joint configurations could be combined to derive a design procedure for the investigated joint configuration. Within the present study, it is assumed that the studied joint is subjected to shear forces only. The torsional effects which could appear at the level of the joint due to the configuration of such beam are prevented as, close to the joint, the C-beam is also connected to bracing members which restrain it against torsion.



(a) 3D view (b) Longitudinal view

Figure 1: Studied connection

3.2 Derived design recommendations and critical reviews of the codes

3.2.1 Preliminary verifications, rotational requirement and ductility conditions

The first point to be checked is the spacing between bolts and edges. According to the thickness of the constitutive elements, this should be verified with EN1993-1-3 or EN1993-1-8, as the requirements regarding the spacing differ in both codes.

Then, a sufficient rotational capacity should be ensured at the level of the joints as the latter should act as a pin. In this regard, reference can be made to the recommendations provided in the codes for header plate and fin plate connections. In particular, (i) the column and the lower flange of the C-beam cannot be in contact, so as to prevent the transfer of compressive forces from the beam flange to the column face and (ii) the available rotation must be greater than the required one. Both conditions lead to the definition of geometrical requirements to be respected between the column face and the bottom beam flange.

Furthermore, ductility conditions have to be fulfilled to ensure a sufficient deformation capacity at the level of the joint components in order to allow for load redistribution within the joints. Accordingly, a premature failure of the bolts should be avoided. In addition, the general requirement reported in Section 5.11 of prEN1993-1-8 has to be satisfied, where it is mentioned, for steel grades up to (but excluding) S460, that a sufficient level of ductility is ensured if the design value of the shear resistance of any fastener $F_{V,Rd}$ is greater than 0.8 times the design value of the bearing resistance $F_{b,Rd}$ obtained with prEN1993-1-8 Table 5.9. For thin-walled sections, this condition is not the same as, in prEN1993-1-3 Table 10.5, it is stated that $F_{V,Rd} \geq 1.2 \cdot F_{b,Rd}$.

Moreover, for the connection between the column face and the angle exhibiting a "header plate behaviour", the resistance of the *bolts in shear* component should be multiply by 0.8 to take into account the effect on the shear resistance of parasite tension force developing in the bolts.

Overall, according to (i) the side of the joint (beam or column) and (ii) the thickness of the connected plates, different ductility criteria may apply. Table 1 details the different requirements adapted to the studied connection to ensure a sufficient level of ductility.

Table 1: Ductility conditions from prEN1993-1-8 and prEN1993-1-3

	Beam side	Column side
Thickness < 4 mm	$F_{V,Rd} \geq 1.2 \cdot F_{b,Rd}$	$F_{V,Rd} \geq 1.5 \cdot F_{b,Rd}$
Thickness ≥ 4 mm	$F_{V,Rd} > 0.8 \cdot F_{b,Rd}$	$F_{V,Rd} > 1.0 \cdot F_{b,Rd}$

The bearing and the shear resistance of any fastener are evaluated in different ways following the side of the joint. For the header plate behaviour, the bearing resistance to be considered for the ductility conditions evaluation corresponds to the maximum resistance between an inner bolt and an end bolt for the case of thick elements. For the case of thin-walled cold-formed elements, as prEN1993-1-3 does not make any difference in terms of bearing resistance for an inner bolt and an end bolt, all the bolts have the same bearing resistance and then the bearing resistance of one fastener is the same for all. Considering the fin plate side, the bearing and the shear resistance should take into account the horizontal and the vertical components of the applied shear forces, and as the end bolt is the most loaded, the resistance of the latter should be calculated as it is governing the resistance of this part of the joint.

In the following sections, rules for the characterisation of the different parts of the joint are proposed.

3.2.2 Column side characterisation

After detailing the ductility criteria, the resistance of the components should be evaluated. Concerning the header plate, six components are activated under shear force. Those ones are listed in Table 2 and the verification formulae that should be used according to both prEN1993-1-8 and prEN1933-1-3 are also referred where available. The resistance of each of the two last components corresponds to the minimum between the bolt shear resistance and the bearing resistance.

Table 2: Activated components from the column side and references to the codes

Components	Reference in prEN1993-1-8	Reference in prEN1993-1-3
L-profile in shear - gross section	C3.3.1(a)	Not available
L-profile in shear - net section	C4.1.1(6)	Not available
L-profile in shear – shear block	5.14	Not available
L-profile in bending	Not available	Not available
Resistance of the column face in bearing	Table 5.6	Table 10.5
Resistance of the L-profile leg in bearing	Table 5.6	Table 10.5

Even if the column side connection is similar to a header plate connection, some differences are however observed, which requires the modification of some resistance equations as proposed in the code. Firstly, the header plate connection as covered in the normative document is bolted with two columns of bolts, one on each side of the connected beam web while, in the studied joint configuration, only one bolt column is used. As a result, the resistance formula should be adapted accordingly.

Then, the formula that computes the resistance of the component *L-profile in shear – gross section*, given by eq. (1), in which a reduction coefficient of 1.27 appears should be modified.

$$V_{Rd,L-profile\ in\ shear-gross\ section} = \frac{1}{1.27} \cdot \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} \quad (1)$$

where,
- A_v : the gross shear area;
- f_y : the yield limit;
- γ_{M0} : safety factor.

Indeed, the resistance of this component is governed by the well-known shear resistance equation with a reducing coefficient of 1.27. In fact, this coefficient aims to decrease the shear resistance due to an M-V interaction existing at the level of the joint, where the bending moment is caused by the shear force multiplied by the lever arm between the bolt column and the mid-thickness of the perpendicular L-profile plate. In a general way, this coefficient has been derived by making the assumption that the plate can reach its elastic bending moment resistance. And, for this level of bending moment in the plate, the resistance in shear has to be divided by 1.27. But, as for the studied configuration, the lever arm is known, by considering the M-V interaction curve, the reducing coefficient 1.27 can be revised, by computing the actual bending moment that is developing in the connected plate, so that the resistance of the component increases.

In addition, it can be observed in the table that no recommendations are given for the shear resistance (gross section, net section and shear block) for a thin cold-formed member. For the resistance of the face in bearing, the column face and the L-profile have to be distinguished when the resistance is computed. In fact, based on the thickness of the constitutive plates, EN1993-1-8 or EN1993-1-3 should be considered as the latter is providing different formulas. During the calculation of this component, the bolts in shear and the bearing resistance, for each fastener, are evaluated and the ductility conditions can be checked. Provided that those are satisfied, if the thickness is lower than 4 mm, the resistance of the fasteners component is obtained with eq. (2) but if the thickness is greater or equal than the limit, eq. (3) should be used.

$$V_{Rd} = n_{bolts} \cdot F_{b,Rd} \quad (2)$$

$$V_{Rd} = \min\left(F_{b,Rd,c,end}; 0.8 \cdot F_{V,Rd}\right) + (n_{bolts} - 1) \cdot \min(F_{b,Rd,c,in}; 0.8 \cdot F_{V,Rd}) \quad (3)$$

where,
- $F_{b,Rd,c,end}$: the bearing resistance of an end bolt row given by EN1993-1-8;
- $F_{b,Rd,c,in}$: the bearing resistance of an inner bolt row given by EN1993-1-8;
- $F_{b,Rd}$: the bearing resistance of a bolt row given by EN1993-1-3;
- $F_{V,Rd}$: the shear resistance of a bolt;
- n_{bolts} : the number of bolts on the column side.

On the other side, if the ductility conditions are not respected, the resistance is computed as the number of fasteners times the lowest design resistance value of any individual fastener.

Finally, no recommendation to assess the resistance of the component *L-profile in bending* is given in prEN1993-1-8. But, this component can be characterised using [3], where equations are proposed.

3.2.3 Beam side characterisation

After determining the activated components from the column side, the beam side, which can be assimilated to a fin plate, is studied where, in total, nine components are activated in shear. Those ones and the associated references in the codes to evaluate their resistance are given in Table 3.

Table 3: Activated components from the beam side and references in the codes

Components	Reference in prEN1993-1-8	Reference in prEN1993-1-3
L-profile in shear – Shear block (beam side)	5.14	Not available
Beam web in shear – Gross section	C4.1.1(7)	Not available
Beam web in shear – Net section	C4.1.1(7)	Not available
Beam web in shear – Shear block	5.14	Not available
L-profile in bending (beam side)	C.4.1.3(5)	Not available
L-profile in buckling (beam side)	C.4.1.2(6)	Not available
Resistance of the fasteners – L-profile plate	Table 5.6	Table 10.5
Resistance of the fasteners – Beam plate	Table 5.6	Table 10.5
Bolts in shear	Table 5.6	Table 10.5

As for the column side, the ductility conditions have to be checked after computing the resistance of the individual fasteners.

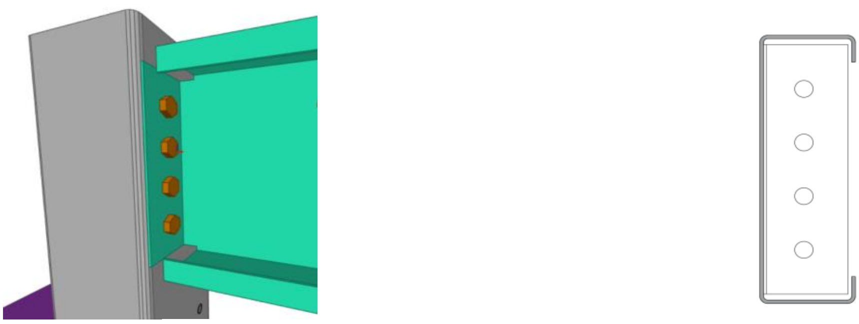
3.2.4 Global design procedure

Once that the resistance of each component has been evaluated, the resistance of the connection is governed by the weakest component according to the component method concept. For sake of safety, only ductile failure mode are targeted. If the failure is linked to a brittle component, then the connection cannot be design and its geometrical and mechanical properties should be modified.

For ease-of-use, a chart summarising the design steps to be crossed for the design of the connection is illustrated in Figure 2.

3.3 Alternative connection

In view of optimisation of the previously investigated joint solution, an alternative connection has been proposed, consisting of an extended part of the C-profile's web, which is bent with an angle of 90° and this part, named "header web", is bolted to the face of the square hollow column, as represented in Figure 3(a).



(a) (b)

Figure 3: Alternative connection

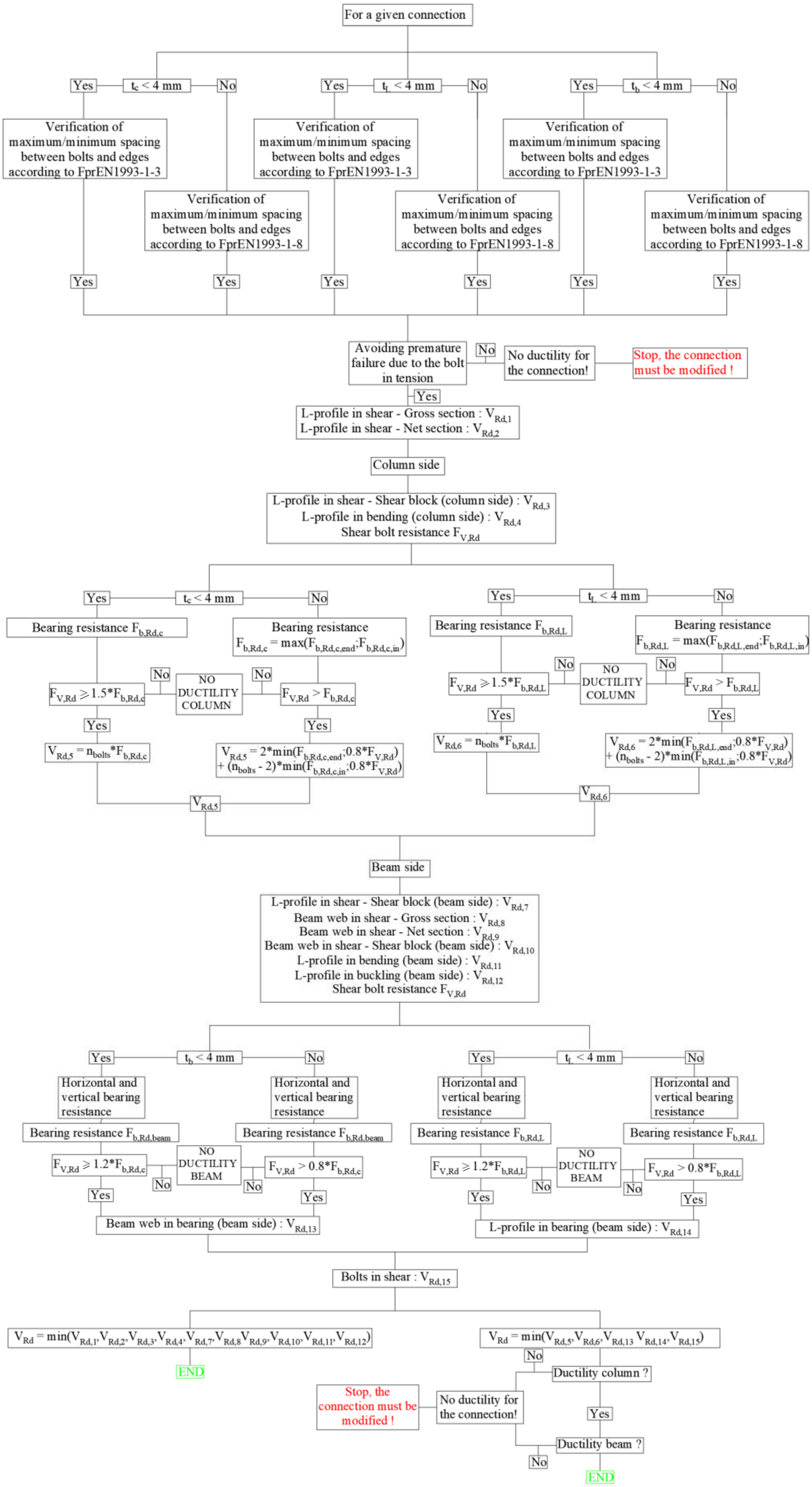


Figure 2: Derived design method

This solution allows avoiding the use of the L-profile piece and limiting the number of bolts. This optimised connection can be seen as the half of a header plate connection and its design is very similar to the one of the column side connection of the previous joint solution.

Five components are activated when the joint is subjected to shear forces namely: (i) *the individual fasteners in shear*, (ii -iv) *the header web in shear (net section, gross section and shear block)*, (v) *header web in bending*. As for the L-profile connection, each equation that characterises the resistance of the components can be found in prEN1993-1-8 annex C, except for the component fasteners in shear, where equations are provided in prEN1993-1-3. It has to be mentioned that the resistance of the extended web, perpendicular to the header web, is taken into account in the component *header web in shear – gross section*, as the geometrical properties are the same.

After investigations and relevant modifications of the available provisions in both codes, a design method has been derived and is presented as a flowchart in Figure 4.

Similarly to the L-profile connection, the same ductility conditions have to be fulfilled. Those conditions must be checked in the same way as for the column part of the previous connection. Figure 4 highlights in blue the ductility criteria and the way to derive the resistance of the fasteners.

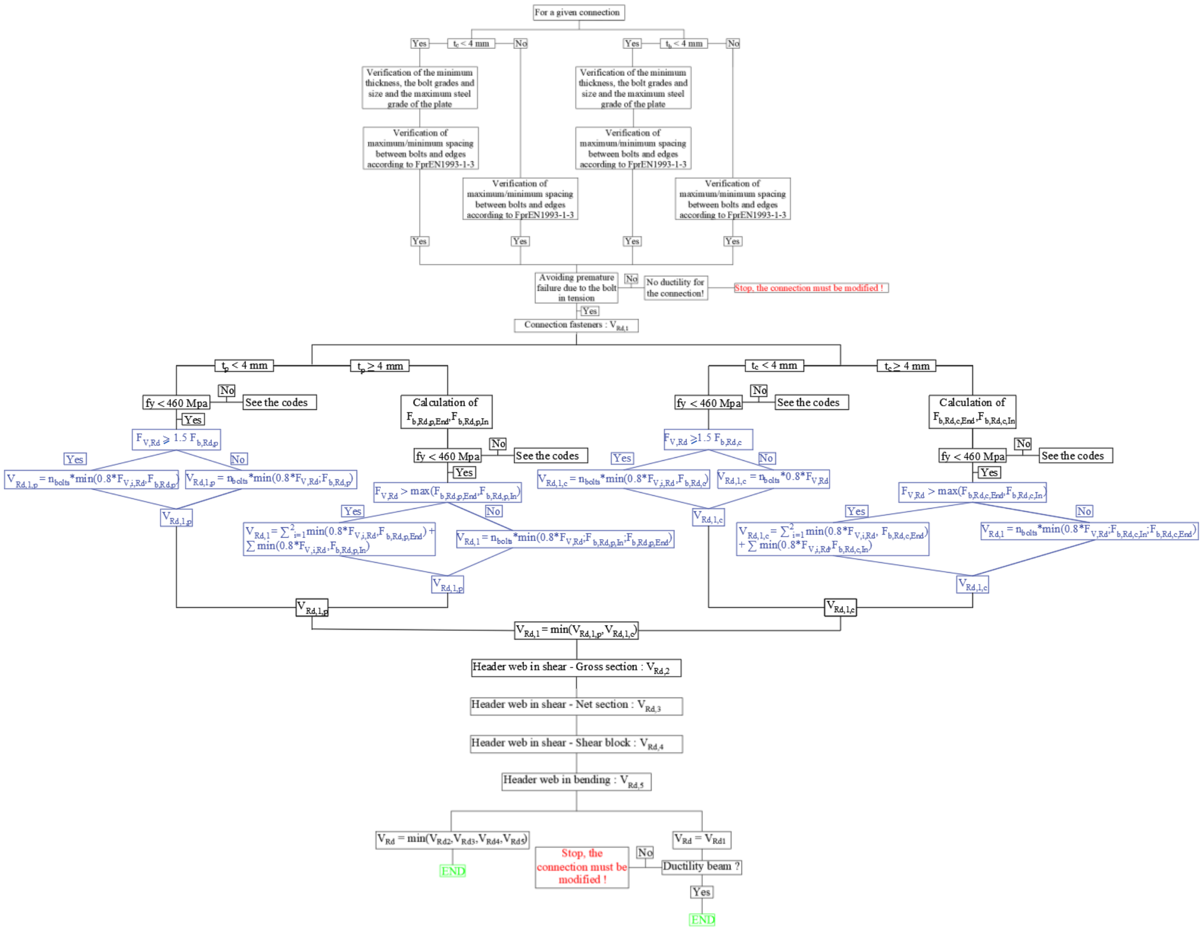


Figure 4: Derived design method for the alternative connection

4. CONCLUSIONS

The present paper focuses on the design method of a pinned beam-to-column joint that is subjected to shear. This joint connects thick and/or thin elements made of cold-formed sections, and especially a C-section beam with a square hollow section column is considered. Through this research, the following outcomes can be highlighted :

- New equations to characterise the bearing resistance are presented in prEN1993-1-8;
- The general normative document about the design of steel joints, EN1993-1-8, does not provide any recommendations for a pinned connection, contrary to the prEN1993-1-8 which gives provisions for pinned joints made of hot rolled thick H or I section elements;
- EN1993-1-3 and prEN1993-1-3 provide the same design equations regarding the resistance of mechanical fasteners subjected to a shear force, which are rather limited to the design of the bolt in shear, the bearing and the resistance of the section in shear;
- The ductility conditions vary depending on the behaviour of the joint i.e. header plate or fin plate and also with the thickness of the connected plates;

Following this first analytical study, an experimental campaign involving tests on both joint configurations as well as numerical simulations are in progress in the framework of the ACTIONS project. Some of the obtained results should be available soon and will be presented at the conference.

5. ACKNOWLEDGMENTS

The authors of this publication acknowledge the Walloon Region, Logistics in Wallonia ASBL and Pôle MecaTech for their support in the framework of the project N°8528 entitled ACTIONS “Advanced Eurocodes Compliant Tools for Industrial Optimized Innovative Structures”. Partners of the project are STOW group, GDTech and CRM companies, as well as the University of Liege.

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