

Full paper

Behaviour of Cold-formed Beam-to-Column Joints in Storage Rack Structure: Experimental Investigations

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Abstract

The rack storage structure market is highly competitive, requiring thus structural design solutions that reduce overall weight. The use of cold-formed profiles is usually adopted for sake of optimisation as the latter are lightweight and can be manufactured in any desired shape. Another potential improvement lies in considering the actual stiffness of beam-to-column joints, rather than assuming them as fully pinned as commonly adopted in the design process. But, considering both solutions requires the characterisation of joints made of thin-walled cold-formed profiles, for which EN1993-1-3:2024 does not provide any recommendation regarding the joint's stiffness. Also, even if relevant provisions are proposed in EN1993-1-8:2024, the latter are only applicable to plates thicker than 3 mm.

This paper presents the results of an experimental test campaign conducted on joints between a thin-walled cold-formed Σ -shaped beam and a square hollow section column, tested in a single configuration commonly met in rack structures. The main objective is to characterize the joint properties to further assess the applicability of the component method provided in the current European standard.

Keywords

Cold-formed, Semi-Rigid, Joints, EN1993-1-3, Storage, Structure

1 Introduction

The worldwide expansion of logistics and warehousing sectors is driving accelerated development of storage infrastructures [1]. With storage density and operational efficiency becoming critical performance indicators, the construction of high-capacity, cost-effective storage rack systems has become a strategic priority for structural rack storage seller. Consequently, design offices are under high pressure to design rack storage solutions that are not only structurally efficient, but also economically competitive. One of the most promising solutions is the self-supported rack structure, which allows to sustain the pallet loads, but also the external actions such as the wind and the snow.

Storage rack systems are predominantly constructed from cold-formed steel members with Σ -shaped thin-walled cross-sections for the beams. Such elements have various advantages, including an advantageous strength-to-

weight ratio, ease of manufacturing and material efficiency. The use of thin-walled elements enables significant reductions in material consumption and transportation costs, thereby allowing the overall cost reductions.

Current practice often involves idealising beam-to-column connections in storage rack structures using simplified joint models that assume either fully pinned or fully rigid connections. In reality, the beam-to-column connections typically exhibit semi-rigid behaviour. Neglecting this semi-rigid behaviour can lead to conservative designs and by consequence, bigger cross-sectional dimensions which is not economically profitable.

The European normative standard EN 1993-1-8:2024 [2] applies to hot-rolled elements with a thickness equal to 3 mm or higher and provides recommendations for assessing the semi-rigidity of a joint. However, the relevant normative standard which applies to cold-formed thin-walled elements, i.e. EN 1993-1-3:2024 [3], only provides

recommendations about joint resistance, without mentioning any provisions on the computation of joint rotational stiffness. By consequence, no recommendations to evaluate the semi-rigid behaviour of connections between cold-formed members are available.

Past research has already investigated beam-to-column joints made of thin-walled cold-formed elements. Bucmys et al. [4] performed a comprehensive study on a C-beam connected to a square hollow section column via a bolted gusset plate for torsional moment force transfer. Based on the experimental results from three specimens, a new 'three-springs' mechanical model was derived for stiffness calculation of such joint. Salih et al. [5] compared the bearing behaviour between thick and thin stainless-steel plates. It was highlighted that bearing of thin plates could lead to curling phenomena which is the out-of-plane deformation of the connected plate in front of the end bolt. Paul et al. [6] performed a complete study on a global frame made of back-to-back C-shaped elements to provide a simplified design method taking account the rotational stiffness of the joints. Experimental, numerical and analytical investigations were conducted. Peng et al. [7] tested 12 apex connections between back-to-back lipped channel with thicknesses included between 1.5 mm and 2.4 mm, to evaluate the rotational stiffness of the apex joint.

This study is focusing on the joint rotational stiffness between a Σ -beam and a square hollow section, both made from cold-formed thin-walled sections. Two specimens were tested in a classical joint configuration that can be met in self-supporting rack structures. For sake of comparison, only the beam thickness was modified between the two specimens. First, the test set-up is presented in section 2, highlighting the specificities of the latter. Then, in section 3, the experimental test results are provided, including the tensile coupon tests results. Finally, section 4 compares and discusses the outcomes of the tests.

This research is part of a Walloon region project, called ACTIONS (convention N° 8528), involving the industrial partners STOW and GDTech, the private research company CRM group and the University of Liège.

2 Experimental test set-up

2.1 Tested sample

The objective of the test campaign is to evaluate the rotational stiffness of bolted joints that connect a thin-walled Σ -beam with a square hollow section (SHS) column. The cross-sectional profiles were fabricated through cold roll forming processes using HX420LAD steel grade. A single assembly type is investigated, namely the continuity connection that corresponds to a continuous beam bolted to the face of the column. The studied configuration is schematized in Figure 1 with the associated nominal geometrical properties of the bolt disposition. The M12 8.8 bolts are placed in a bolt hole of 12.5 mm diameter. For tightening the connector, a handhole is cut in the perpendicular face of the column. The thickness of the SHS column is 2.5 mm while that of the beam is either 2 mm or 2.5 mm. The beam profile is named $\Sigma 200/2.0$ and $\Sigma 200/2.5$, respectively.

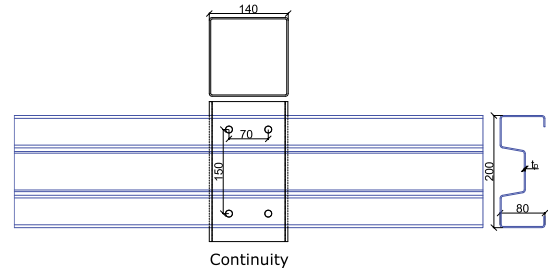


Figure 1 Tested joint configuration, in [mm]

2.2 Test set-up presentation

The experimental test set-up was designed so that the targeted failure mode corresponds to the bearing of the thin-walled beam web or column web, as this one is still not well mastered from a normative point of view, as explained before. For that, the buckling resistance of both elements, the bolts in shear resistance, the column web panel in shear resistance and the cross-sectional resistance were characterised to avoid these failure mechanisms. Regarding the beam cross-section, its flexural resistance, computed according to EN1993-1-3:2024, is equal to 17.59 kN.m and 23.4 kN.m, for the $\Sigma 200/2.0$ and $\Sigma 200/2.5$, respectively.

The test set-up configuration is provided in Figure 3. The beam is connected with 4 bolts to the mid-height of the 1,500 mm column. Boundary conditions were established such that the structural system represents a column with a pinned support at its upper extremity, while the lower pinned support extremity allows longitudinal displacement along the Y-axis (see Figure 2) to prevent parasitic internal forces arising from axial shortening of the column. Regarding the beam, it can be assumed as a cantilever element. At the left free extremity, the beam is attached to the jack with a mechanical device that allows the in-plane rotation of the latter. In addition, at the Σ cross-section on which the load is applied, stiffeners are installed to avoid any local buckling due to the punctual load. As the load is applied with an eccentricity from the shear centre, a torsional moment in the Σ -beam is created. Thus, out-of-plane supports were positioned at 1,110 mm horizontally from the gravity centre of the joint. Furthermore, the torsion of the column around its longitudinal axis was prevented through these supports. Accordingly, these supports can be assumed as working as fork supports. The load was applied at a horizontal distance from the gravity centre of the joint equal to 1,717 mm. It should be noted that for the stability of the jack, the system was inverted so that the jack pulls on the Σ -beam. The static configuration scheme of the test set-up is presented in Figure 2.

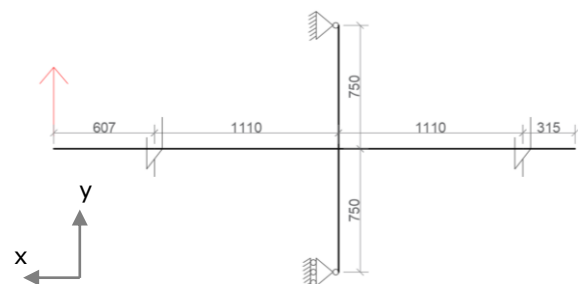


Figure 2 Test static scheme, in [mm]

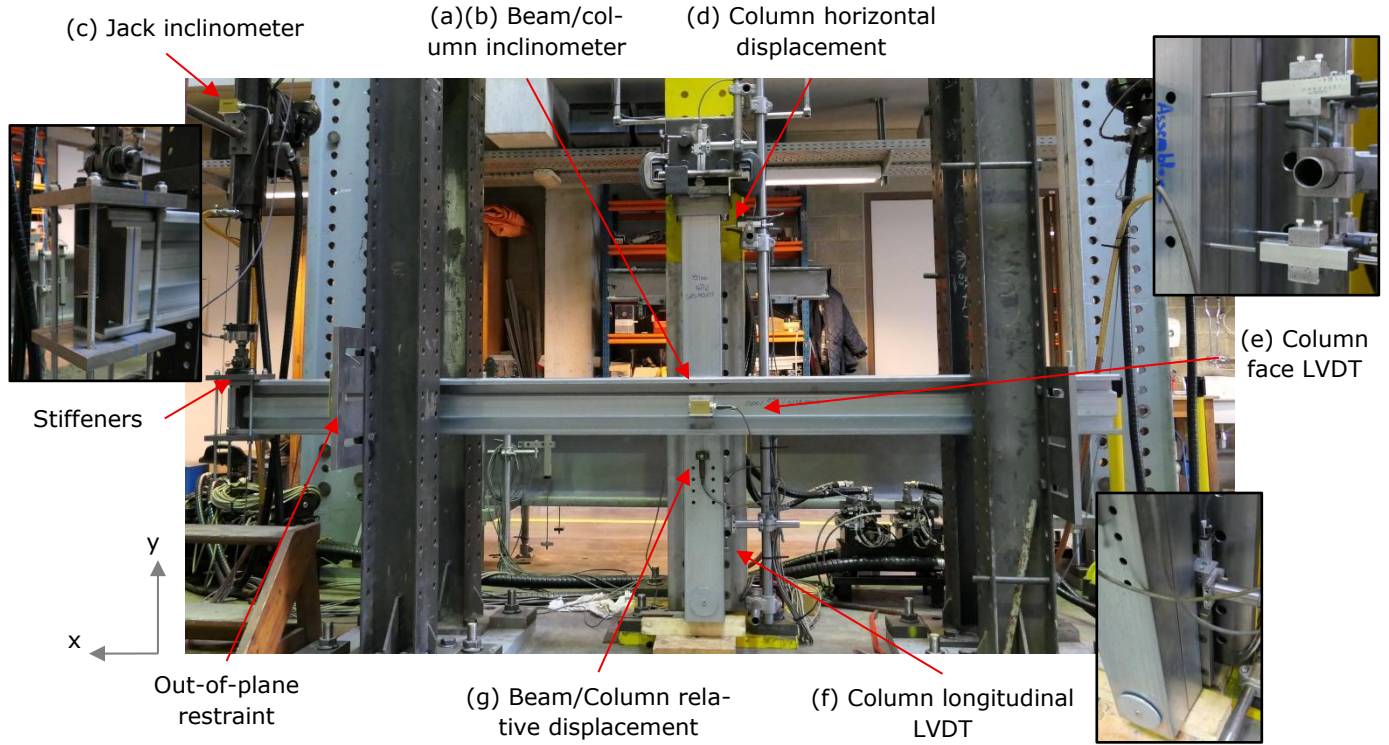


Figure 3 Test set-up configuration

Regarding the instrumentation of the set-up, one inclinometer was positioned at the bolt group gravity centre on the Σ -beam ((a) see Figure 3) and also on the column face (b), inside the column. A third rotational measurement sensor (c) was placed on the jack, to further evaluate the inclination of the applied load during the test. In addition, seven Linear Variable Differential Transformer (LVDT) were installed on the set-up. The first one (d) measured the horizontal displacement at 100 mm from the fixed column cross-section extremity (upper support in Figure 3), so that the rotation can be derived. Then, a group of four LVDTs (e) measured the shear deformation of the column face. One LVDT (f) recorded the vertical shortening of the column and the last one (g) measured the relative vertical displacement between the beam and the column.

3 Experimental test results

3.1 Tensile coupon test results

The material mechanical properties of the steels from the different constitutive elements of the tested specimens were characterized through tensile coupon tests. One dog-bone specimen was extracted longitudinally from the beam and from the column. The proceedings of the test follow the ISO 6892-1 [8] standards. The obtained engineering stress-strain curves are reflected in Figure 4 while the main analytical values are reported in Table 1.

Table 1 Coupon tests mechanical properties

Data	$\Sigma 200/2.0$	$\Sigma 200/2.5$	SHS140/2.5
R_{eH} [MPa]	500	525	533
R_m [MPa]	575	578	588
E [MPa]	182,853	207,442	200,604
A [%]	22.76	22.52	22.6

Comparing the mechanical properties obtained through the tensile tests with the nominal properties provided in EN1993-1-3:2024 ($f_y = 400$ MPa, $f_u = 450$ MPa and $E = 210,000$ MPa), the former are higher than the latter, except for the Young Modulus. Moreover, even if all the elements are made from the same steel grade, slight differences can be observed.

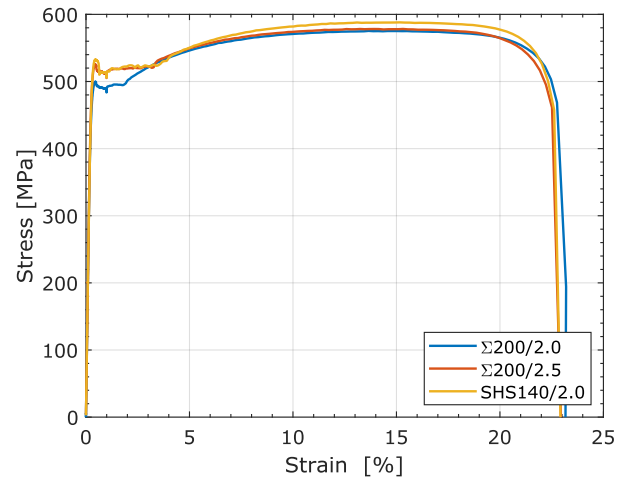


Figure 4 Experimental engineering stress-strain curves

3.2 Test results

Prior to the beginning of the tests, the bolts were slightly preloaded with a torque value equals to 25 N.m in order to master this parameter during the analysis of the results and the validation of the numerical models. The bending moment applied at the joint was computed according to the applied load times its lever arm. The ultimate applied force by the jack F_u , the corresponding bending moment M_u , the ultimate rotation of the beam ϕ_u and the jack shortening Δ_j are provided in Table 2. The graph that links

the rotation of the Σ -beam (measured through the inclinometer on the Σ -beam) to the applied bending moment is provided in Figure 5, for $\Sigma 200/2.0$ and $\Sigma 200/2.5$ profiles.

Table 2 Joint continuity configuration - Experimental test results

	F_u [kN]	M_u [kN.m]	ϕ_u [rad]	Δ_j [mm]
$\Sigma 200/2.0$	8.18	14.04	0.169	305.25
$\Sigma 200/2.5$	9.29	15.95	0.198	354.28

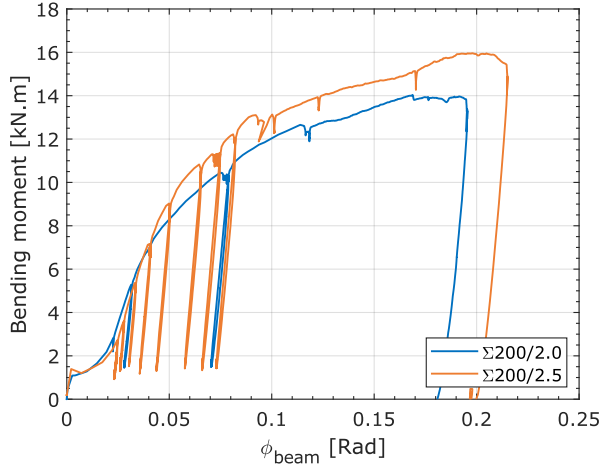


Figure 5 Experimental test results – $M - \phi_{beam}$ curves

The graph can be divided in four different phases; (i) linear behaviour up to an applied bending moment between 1 and 1.5 kN.m – during this phase, the load is transmitted from the beam to the column through friction. In the second (ii) phase, the slope is almost horizontal – this corresponds to the relative movement of the beam compared to the column, so that the bolts came in contact with the bolt hole perimeter to transmit the load through bearing actions. The third (iii) phase represents the main behaviour of the joint and corresponds to the bearing actions during which the load is transferred from the beam to the column through bearing. As already demonstrated in previous research [9], this relation is purely non-linear, because as soon as the bolt is in contact with the bolt hole, yielding occurs as all the force is transmitted in one point, resulting in a high stress concentration that exceeds the yield limit of the material. Finally, the last phase (iv) occurred for high intensity of the applied bending moment for which the deformation is such that 2nd order effect appeared and part of the load is transmitted through axial force in the beam. This can explain the end of the curve that exhibits a slightly higher slope, which mean that a higher rigidity is activated in the system.

The obtained failure mode is related to the bearing of the beam for both beam thicknesses, as illustrated in Figure 6 for the $\Sigma 200/2.5$ beam. The bearing deformation pattern develops perpendicular to the radius extending from the bolt group centroid to the individual bolt hole, which is in line with the expectations since this corresponds to the resultant force acting on each fastener.

The relation between the applied moment and the rotation of the column face is provided in Figure 7. It should be noted that the column remains in its elastic domain, as the relation is perfectly linear. This emphasises the fact that the failure appeared in the beam. In addition, the rotation of the column is limited with a maximum of 0.011 rad for

the thinner beam profile, and 0.01 rad for the thicker one.

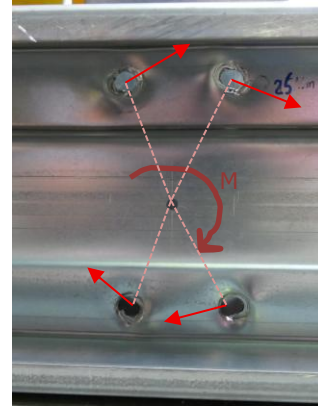


Figure 6 Bearing deformation in the $\Sigma 200/2.5$ beam at the failure

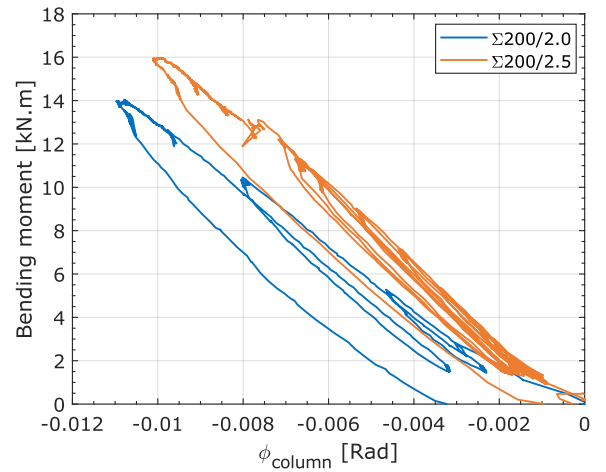


Figure 7 Experimental test results – $M - \phi_{column}$ curves

The recorded measures from the four LVDTs located on the perpendicular column face, aiming at studying the column face in shear, are presented in Figure 8 in which the continuous line represents the $\Sigma 200/2.0$ beam behaviour and the dashed line the $\Sigma 200/2.5$ one. The displacement is negative in the direction of the X-axis (see Figure 2).

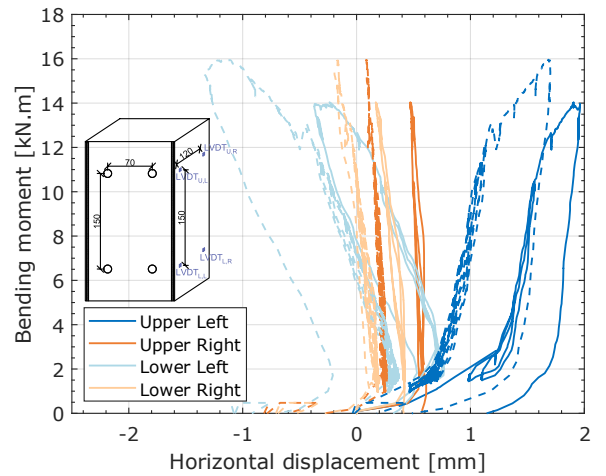


Figure 8 Deformation of the column face according to X-axis, $\Sigma 200/2.0$ continuous line and $\Sigma 200/2.5$ dashed line

Considering only one beam thickness, the graph shows that the LVDT on the right side recorded almost no displacement while the behaviour of the left side sensors between the upper and the lower one is opposite. This can be explained by the shear deformation of the web column panel, in combination with the one due to the global bending deformation of the column. This is schematically explained in Figure 9, in which the green force represent the shear resultant R_H from the torsional bending moment M , the yellow one the bending moment displacement of the column and the pink arrows are related to the recorded direction of the deformation.

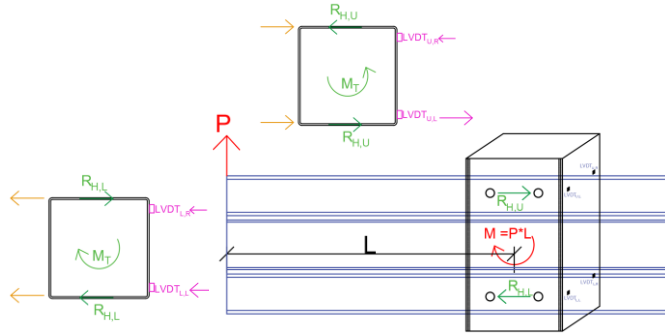


Figure 9 Column face shear deformation – force distribution

At specified applied load intensities, unload-reload cycles were applied, in order to characterise the stiffness of the joint according to the applied bending moment. As can be seen in Figure 5, all the unload-reload lines are parallel, meaning that the joint stiffness remains constant during the experimental test. The mean stiffness value for the $\Sigma 200/2.0$ beam, including the final unload, is equal to 1,039.13 kN.m/rad with a standard deviation of 48.28 while for the thicker profile, the mean equals 1,110.59 kN.m/rad with an associated standard deviation of 38.62.

Finally, the maximum rotation of the column at the fixed support (upper extremity) is equal to 6.98 mrad and 8.5 mrad, for $\Sigma 200/2.0$ and $\Sigma 200/2.5$ beams, respectively. The shortening of the column, recorded by the LVDT, is almost negligible, which is in line with the expectation as the axial load in the column is small compared to its axial rigidity.

4 Comparison and discussion

A single connection detail with two distinct beam profile thicknesses was tested. As expected, the thicker profile reached a higher resistance as the tests were designed to activate the beam web in bearing as failure mode. The ratio between the two ultimate resistance is $\frac{M_{u,\Sigma 200/2.5}}{M_{u,\Sigma 200/2.0}} = 1.136$ and the ratio between the ultimate rotations equals 1.171. Both joints exhibit the same behaviour and the failure appeared due to the bearing deformation in the bolt holes of the Σ -beam, as expected.

But the achievement of the ultimate resistance for both tested specimens is associated with significant rotations and deformations of the specimens which are not acceptable in the framework of a rack structure design.

In terms of stiffness, it has been observed that, during the

first loading, the joint behaviour is fully non-linear as justified above while, during unload-reload cycles, the behaviour is linear.

Accordingly, the design resistance of such joint should be defined in such a way to limit the local deformation at the level of the joint while, for the stiffness, a method to predict the joint stiffness observed during the unload-reload phase should be developed as this stiffness is the relevant one when performing rack structure analyses. These aspects constitute perspectives of development to propose relevant design criteria for such joints.

5 Conclusion

This paper reports about investigations on the behaviour of beam-to-column connection between a thin-walled cold-formed Σ -beam and SHS column with the final objective of taking into account the rotational joint stiffness in the design of a rack storage structure. To achieve this, two joints with the same configuration were tested, varying the thickness of the Σ -beam cross-section.

The experimental investigations showed that:

- Ultimate resistances of 14.04 kN.m and 15.95 kN.m were reached for the thinner and the thicker beam profiles, respectively;
- An excessive bearing deformation occurred in both Σ -beam, leading to the failure of the joint;
- The joint stiffness remains constant during the unload-reload cycles. The mean value is equal to 1,039.13 kN.m/rad and 1,110.59 kN.m/rad, for $\Sigma 200/2.0$ and $\Sigma 200/2.5$, respectively;
- Compared to the beam rotation, the one associated to the column face is negligible and remains in its elastic domain throughout the test;
- The column face deformation is due to two contributions: (i) the horizontal resultant of the torsional moment of the bolt group and (ii) the global bending moment deformation of the column.

This conference paper outlines preliminary results from the experimental test campaign. Additional research will be undertaken, primarily through comparisons of the experimental results with the component method predictions as proposed in the European standard. The final objective is to develop a practical analytical design procedure that enables practitioners to efficiently determine the resistance and the rotational stiffness of the investigated joints when designing such steel structures. Further results will be shared during the oral presentation at the conference.

This research takes place in the framework of the Walloon Region project with the acronym ACTIONS studying the behaviour of rack structures. Relevant outcomes of this project can be found in [10]-[13].

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