Ductility improvement of adjustable pallet rack speed-lock connections: Experimental study

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1. Introduction

Adjustable pallet-rack systems are heavy-duty steel-shelved structures intended to store goods. Fig. 1 describes pallet-rack systems. Fig. 1a presents a general view of such a structure; it is composed of vertical (uprights), horizontal (beams), and sloping (braces) bar-like elements. Fig. 1a shows that the longitudinal horizontal direction is termed as down-aisle while the transverse one is referred as cross-aisle. Commonly, all the structural members are made of thin-gauge cold-formed steel profiles. Speed-lock upright-to-beam connections are employed to facilitate the rack erection, being based on inserting hooks into upright perforations (tabs), as described by Fig. 1b.

The seismic design of pallet-racking systems is a relevant issue, given their significant vulnerability and the high seismic hazard of many sites [1,2]. The rack’s vulnerability is mainly contributed by their low lateral strength and stiffness and their important live load masses. The live masses are highly variable, randomly distributed, and can slide on the rack; these uncertainties prevent precise estimations of the modal parameters. Regarding the lack of lateral capacity, it can be considered as more critical in the down-aisle direction, given the usual absence of bracing [3,4]; actually, in some cases, only rear bracing is provided (Fig. 1a), thus generating undesired twisting motion. Accordingly, this paper deals with the seismic performance of adjustable pallet-rack systems in the longitudinal (down-aisle) direction. One of the factors (but not the only one) that contributes to the lateral flexibility is the looseess and low stiffness of the beam-to-upright connections. Fig. 1b shows such a connection, where two beams are framed at both sides of a continuous upright. Each beam is welded to an end-plate; in its turn, such a plate is connected to the upright through several hooks inserted into the upright perforations. Given the unavoidable gap between the hooks and the perforations, it is obvious that these connections exhibit relevant slippage (looseness) and are rather flexible [3], as previously announced. In other words, under down-aisle seismic shaking, the beam-to-upright connections (and the foundation-upright connections)
are the weakest and most flexible parts; therefore, their stiffness and energy-absorption capacity are of primary importance to the rack overall seismic resistance. Moreover, the main structural members (mainly the uprights and the braces) are of class 4 [5] (i.e. slender, according to American documents), while the beams can be either of classes 3 or 4 (semi-compact or slender). Hence, energy dissipation cannot rely on the main structural members. This trend makes that the rack behavior factor needs to be established mainly based on the characteristics of the connections.

Given the circumstances discussed in the previous paragraph, important research activity on the seismic design of racks has been undertaken worldwide. Broadly speaking, two major approaches have been proposed for the seismic design of pallet racking systems, namely dissipative and non-dissipative concepts [6]. In the non-dissipative design, little or no damage is accepted; conversely, in the dissipative design, only the overall structural integrity is pursued and, thus, greater damage is accepted. In other words, in the non-dissipative approach, the structure remains in its linear range and, thus, no energy is absorbed; in the dissipative approach, the opposite occurs. The major pros and cons of both design solutions are briefed in the next paragraph.

In the non-dissipative strategy, costly rigid and robust structures are to be designed; on the contrary, in the dissipative case, more economical and less robust racks can be considered. However, such racks need to be sufficiently ductile, and, moreover, relevant damage is to be expected after severe seismic events, thus generating higher repair and replacement costs.

The non-dissipative approach is commonly employed nowadays, perhaps due to the rather low ductility of racks and a certain scarcity of experimental and theoretical studies on their nonlinear behavior. On the contrary, in the seismic design of civil engineering constructions, the dissipative approach is routinely considered. As discussed later in this section, this research considers the dissipative option.

Regarding more advanced seismic design issues, the work [7] proposes a novel formulation that combines nonlinear dynamic analyses with assessing the cumulative damage in the beam-to-upright connections (thus accounting for fatigue). The later paper [8] utilizes this approach to perform a parametric study of the seismic capacity of a set of representative racks. This latter study shows that the common simplified seismic design approaches are rather inadequate for racks. Also, the research [9] presents nonlinear static (pushover) and Incremental Dynamic Analyses (IDA) for rack structures and compares their results, concluding that pushover is not always on the safe side. Moving to a different approach, the work [10] proposes a methodology for the seismic vulnerability assessment of steel racks in terms of fragility curves. Additionally, [11] deals with a new structural design solution based on seismic (base) isolation of racks. Finally, [12] compares different novel seismic devices for steel storage structures, mainly based on seismic isolation and energy dissipation.

This paragraph deals with the influence of the plate-to-beam weld path in the beam-to-upright connection ductility. Two major types of bending failure of the beam-to-upright connections are possible (Fig. 1b): first, the rupture of the top or bottom ends of the welds between the end-plate and the beam; and second, the sliding of the hooked assembly between the upright and the end-plate; these two failure modes are rather brittle and ductile, respectively. Such failures are illustrated later in Fig. 9a and b, respectively; their occurrence depends on the plate-to-beam weld beads geometric configuration and the hooked assembly strength. Given that this strength commonly degrades during severe seismic excitations, the failure type might depend on the testing protocol. As ductile failures provide relevant advantages, a higher resistance of the welding configuration is preferred. Noticeably, this situation is similar to the classical “strong column - weak beam” recommendation in the seismic design of buildings. In that case, the European regulations [13] state that the column over-strength is 1.3; given the high uncertainty in the resistance estimation (by theoretical or experimental means) of both failure modes, a similar or higher margin may be advisable for beam-to-upright connections. In other words, a relevant welding over-strength is recommended. On the other hand, if a connection completely fails so that the corresponding beam falls, two undesired effects are generated: the subsequent lack of upright buckling restraint usually leads to its immediate collapse, and the fallen pallets could collide with another part of the rack. In most cases, this leads to the collapse of the rack [14].

Finally, this paragraph describes the scope of this research. As previously announced in this section, this paper deals with the dissipative approach, given the potential advantages of such methodology; in fact, this work is a part of a wider research effort aiming to develop and promote this design strategy. As discussed above in this section, there is a strong need for testing, mainly the components where most of the energy is dissipated; therefore, this research activity includes testing campaigns on the beam-to-upright and the upright-base plate connections [15,16]. The results of these experiments are to be compared with

![Fig. 1. Analyzed racking systems.](image-url)
numerical simulations in order to calibrate the employed numerical models. As well, these studies will provide expressions of the response modification factor $R$ in the American practice \cite{17,18}, being known as ductility behavior factor $q$ in the European regulations \cite{19,20}; such expressions might be employed in simplified code-type design strategies \cite{21}. This paper describes the first step of this research, namely the experiments on the beam-to-upright connections. The study focuses on the beam-to-plate welding strength, for reasons discussed earlier in this section.

2. State-of-the-art of testing of beam-to-upright connections

Several previous test campaigns on this issue have been reported \cite{11,22-33}. The most recent studies related to the presented research are listed and discussed next:

- \cite{Roure et al. 2013}. This document \cite{27} compares tests of cantilever boltless (clip-on type) beam-to-column connections performed according to European and American regulations. Significant differences are observed.
- \cite{Zhao et al. 2014}. This paper \cite{28} refers to monotonic downward (hogging) experiments of beam-to-upright speed-lock connections. The influence of the beam geometrical sectional parameters is analyzed. The tests are conducted according to the European regulation \cite{5} and the American document \cite{17}.
- \cite{Yin et al. 2016}. This work \cite{29} describes monotonic and cyclic experiments on speed-lock beam-to-upright connections; both bolted and bolt-less connections are tested. It is concluded that the bolts improve the connection performance, although they certainly impair its speed-lock character. Two weld beads geometric configurations between the beam and the end-plate are considered: the first, along both lateral sides of the beam, and, the second, along its whole perimeter (all around). The experiments are performed according to European \cite{5} and American regulations \cite{34}.
- \cite{Giordano et al. 2017}. This study \cite{30} reports monotonic and cyclic tests on beam-end connectors of cold-formed steel storage pallet racks. Similar to \cite{29}, two weld configurations between the beam and the end-plate are considered: lateral sides and all around.
- \cite{Dai, Zhao, Rasmussen 2018}. This article \cite{31} presents cyclic cantilever tests of bolted beam-to-upright connections; the tested specimens differ in the upright thickness, beam height, and number of tabs and bolts. The influence of these parameters is discussed, and comparisons with boltless connections are performed; also, the authors propose using the so-called Pinching$^4$ model in OpenSees.
- \cite{Gusella et al. 2018}. This paper \cite{32} presents monotonic and cyclic tests on both bolted and boltless beam-to-column joints of industrial pallet racks. The tested joints differ in the type of beam-connector (with different welding layouts), the number of tabs, and the relative thickness of the upright and the beam-end connector; the key role of welding in the failure mode is remarked. Significant pinching is identified.
- \cite{Giorglioni (2016) book}. This book \cite{2} proposes a new protocol that might reproduce more accurately the actual seismic behavior of loaded racks.

3. Experimental campaign

3.1. Objectives of the experiments

The tests mentioned in Section 1 refer to the hysteretic bending behavior of beam-to-upright connections. With this aim, seismic monotonic and cyclic proofs are intended. The need for these proofs has been discussed previously in Section 1. It should be emphasized here that the hysteretic behavior of the connections cannot be analyzed exclusively by numerical simulation since the connecting elements are highly irregular, thus generating sliding, uneven contact, localized yielding, and other complex phenomena. Therefore, the objectives of the carried-out experiments are: (i) to investigate the behavior (capacity and ductility) of the tested connections, (ii) to propose design alternatives, (iii) to develop incremental and hysteretic constitutive laws (for push-over and dynamic analyses, respectively), and (iv) to compare with both simplified and more complex numerical models (based on Strength of Materials and on Continuum Mechanics theories, respectively). The experiments presented in this paper are designed to achieve such objectives and, as discussed previously, are planned after the studies listed in Section 1. Compared to these previous tests, major novelties are: (i) consideration of several alternative weld paths (beads geometric configurations between the beam and the end-plate), (ii) contemplation of different criteria to define (shape) the proofs \cite{2,35}, and (iii) comparison between different cyclic testing bounds. Moreover, it should be emphasized that only speed-lock connections (bolt-less) are analyzed. The next subsection discusses these issues more deeply.

3.2. Tested specimens

As briefly introduced in Section 1, the experiments consist in loading vertically (until failure) beam-to-upright connections, like the one portrayed in Figs. 1b, 2 describes more specifically the testing specimens. Fig. 2a displays a global image view, showing that each sample is T-shaped, being composed of a segment of an upright and a cantilevered segment of a beam. Fig. 2b displays a closer zoom view of the connection itself. The specimens are full-scale, the upright and beam elements being selected among common commercially available cold-formed products. Their sections are displayed in Fig. 2c and d, respectively. The upright is 500 mm long, and its section is Omega-shaped (“Cee” shape with returns); the plate thickness is 2.5 mm, and the section is 69 mm deep and 122 mm wide. The end-plate (angle beam-end-connector) is 3.5 mm thick; 210 mm long, and their sides are 40 (xz plane) and 60 (yz plane) mm. The beam is 600 mm long, and its section is formed by assembling two C-lipped channel profiles (highlighted in red and purple in Fig. 2d); their thickness is 1.5 mm, and the section is 110 mm deep and 50 mm wide. As discussed previously, Fig. 1b shows that the beam is welded to the end-plate, which is connected, in turn, to the upright through hooks (studs) inserted into the upright holes.

Four options for the beam-to-upright connection are tested; as
Fig. 2. Tested specimens.

Fig. 3. Beam to end-plate weld beads configurations.
suggested by the recent previous researches discussed in Section 1, they differ merely on the weld between the beam and the end-plate:

- Welding along the vertical (lateral) sides. This option is named VS (Vertical Sides) in this paper.
- Welding along the whole perimeter (vertical and horizontal sides). This solution is denoted as FP (Full Perimeter). This welding affects only the external flanges of the beam, while the internal ones are not directly connected to the end-plate (red in the top and purple in the bottom, Fig. 2d); in other words, the weld chord gets only the external parts of the assembled shape.
- Welding along the vertical (lateral) and the top sides (like FP although without welding the bottom side). This solution is named IU, as the geometric configuration resembles an inverted U.
- Welding along the whole perimeter and, unlike the FP and IU options, also connecting the internal flanges of the beam to the plate (double-sided welding); that welding is performed from the outside. This case is denoted as FFIP (Full Perimeter and Internal Flanges).

Fig. 3a–d display, respectively, images and sketches of these four connection options; the sketch in Fig. 3d describes how the C-shaped channel profiles are cut in order to allow the weld to connect the inner and outer flanges (even the folded ends) to the end-plate.

The first welding option (Fig. 3a) refers to the unmodified commercially available product, while the other (Fig. 3b–d) incorporate alterations aiming to strengthen the plate-to-beam welding. Noticeably, all these variations maintain the speed-lock character of the connection.

The steel grade for the uprights and the end-plates is S355 MC [36] and S275 [37]; apart from these nominal values, the actual mechanical parameters of the steel are determined after coupon tests. Also, the plate thickness is experimentally measured. In the performed tests, steel from five different coils has been utilized; Table 1 displays the experimental mechanical and geometrical parameters for each of them.

The important differences shown in Table 1 between the nominal (coil) and measured (element) yield stresses can be explained by the alleged excess of strength in the coil itself; conversely, the yield stress increase due to the folding process is not relevant, as the coupon specimens are taken from the flat parts of the section. Moreover, coils 1 and 2 were re-rolled to obtain the desired thickness; hence, in such a case, the yield stress increase is higher. Another consequence is the important brittleness attained: \( f_y \) and \( f_u \) are very close for beams; this low margin is not in accordance with the seismic design standards. Nonetheless, yielding occurs in the end-plate and the upright tabs, not in the beam profile. In fact, the objective of this research (strengthening the weld) is to avoid the brittle failure of the beam. Finally, a notable Young Modulus scattering between coils is observed; this issue is expected to influence the initial stiffness of the connection.

### 3.3. Testing set-up

Fig. 4 describes the testing mock-up; Fig. 4a displays a global image view, and Fig. 4b presents a side sketch.

Fig. 4 shows that the upright and the beam segments are positioned (like in actual racks) in vertical and horizontal directions, respectively; their support conditions are described next. The upright segment is pinned at its top and bottom ends; all displacements are restrained at both ends. The beam segment’s right end is free to rotate about \( z \) and \( y \) axes (bending), while the \( x \)-axis rotation (torsion) is prevented.

As discussed more deeply in Subsection 3.4, the experiments consist basically in imposing, by using the right jack (Fig. 4a), vertical displacements to the right end of the beam segment; both downward and upward displacements are applied. The left jack slightly compresses the upright segment permanently to avoid uplift when the beam is pushed up (Fig. 4a).

Two major sets of sensors are employed: an assembly of two horizontal displacement transducers to measure the end-plate rotation about \( y \) axis (\( \theta_y \)), and a load cell to measure the jack force.

### 3.4. Conducted experiments

As said above, two types of experiments are carried out: monotonic tests and cyclic tests. For both, regarding the particular regulations for adjustable pallet racking systems, standard [5] is contemplated for general testing set-up issues. As well, the directions of the Eurocode 3 for constructional steel are also broadly accounted for; more precisely, [38] contains general rules and [39] deals with joints. The monotonic and cyclic tests are broadly discussed next.

- Monotonic tests. Both downward (beam hogging bending) and upward (beam sagging bending) monotonic proofs are carried out; their main objective is to define the limits of the imposed displacement laws of the cyclic experiments. Such cyclic bounds can be selected either using the racking code [20] or the generic regulation for connections [35]; the latter document is considered in this research.

  On the other hand, as indicated by the regulations [20], a minimum of three tests are performed in downward and reverse directions. Finally, Fig. 4b shows that an initial constant descendent vertical force of 5 kN is applied to the beam near its connection with the upright; this force is exerted through a lever mechanism, as displayed by Fig. 5. Such 5 kN load has several objectives: (i) to represent somehow the shear force on the connection due to the gravity effect of the stored goods, (ii) to reproduce approximately the proportion between shear force and bending moment in actual connections, (iii) to avoid the undesired influence of gap, and (iv) to prevent hook slippage in upward tests. The value of this load is taken from [20]. In brief, all the considerations about pallet racks to define the monotonic tests are based on [20], while the cycling bounds (limits) for the subsequent cyclic tests have been taken according to [35].

- Cyclic tests. Two versions of cyclic tests are carried out: some are based on [35], while others consider the alternative testing method described in [2]. In both cases, the cycling bounds are selected according to the monotonic proofs shaped as [35], while the test arrangement follows [20]. As a consequence of this last issue, the 5 kN load mentioned above is also applied. Such load is not considered either in [35] or in [2]; this absence can be explained as [35] is intended to general steel connections (the issues discussed in the previous paragraph do not apply), and [2] does not deal with full

<table>
<thead>
<tr>
<th>Coil no.</th>
<th>Element</th>
<th>Thickness (mm)</th>
<th>Yield stress ( f_y ) (MPa)</th>
<th>Ultimate stress ( f_u ) (MPa)</th>
<th>Young Modulus ( E ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Nominal</td>
<td>Mean</td>
<td>Charact.</td>
<td>Nominal</td>
</tr>
<tr>
<td>1</td>
<td>Beam</td>
<td>1.5</td>
<td>1.483</td>
<td>1.46</td>
<td>275*</td>
</tr>
<tr>
<td>2</td>
<td>Beam</td>
<td>1.5</td>
<td>1.513</td>
<td>1.49</td>
<td>275*</td>
</tr>
<tr>
<td>3</td>
<td>Upright</td>
<td>2.5</td>
<td>2.55</td>
<td>2.55</td>
<td>355</td>
</tr>
<tr>
<td>4</td>
<td>End-plate</td>
<td>3.5</td>
<td>3.50</td>
<td>3.50</td>
<td>355</td>
</tr>
<tr>
<td>5</td>
<td>End-plate</td>
<td>3.5</td>
<td>3.543</td>
<td>3.39</td>
<td>355</td>
</tr>
</tbody>
</table>

* The values of \( f_y \) and \( f_u \) were increased by cold re-rolling.
speed-lock connections (since a locking bolt is placed, thus avoiding the gap between hooks and perforations, and hook slippage).

Table 2 describes the main characteristics of the carried-out experiments. The tests’ names, VS, FP, IU, and FPIF, stand for Vertical Sides (Fig. 3a), Full Perimeter (Fig. 3b), Inverted U (Fig. 3c), and Full Perimeter and Internal Flanges, respectively (Fig. 3d). M and C refer to Monotonic and Cyclic, respectively. In the monotonic tests, – and + correspond to upward and downward directions, respectively; tests FPIF-A and FPIF-B merely differ in the employed steel coil. In the cyclic tests, ECCS and CAS indicate the norm or document used to state the imposed displacement law ([2,35], respectively). The IU weld configuration is employed in the CAS tests, as the bottom horizontal side is not expected to work under tension; therefore, the IU specimens should be expected to perform like the FP ones. The test FPIF-C-0 was developed according to [2] without using the 5 kN force (Fig. 5) and performing only the four initial loading cycles. Finally, the replica tests are distinguished by adding (1,2,3) to their name.

Table 2, the FPIF-C-ECCS tests performed on 03/05/2018 (I) and 21/11/2017 (II) were conducted by using different cycle bounds than in the other similar proofs; specifically, the former used the bounds of FP tests, and the latter used a different positive limit ($\theta_y^+ = 15$ mrad instead of $\theta_y^+ = 11.7$ mrad, Table 3); as stated in Table 2, such tests are referred next as FPIF-C-ECCS-I and FPIF-C-ECCS-II, respectively. This action aims
previously, such information is employed to establish the bounds of the cyclic experiments displacement laws. In this sense, Fig. 6 displays the reason is that, for such type of test, early failure was detected in all the discussed in Subsection 4.2.

Table 2 shows that two displacement protocols are imposed in the cyclic tests, namely according to [2, 35]. Both documents consider four increasing cycles and several sets of three constant inelastic cycles, whose amplitude is increased until failure. Precise descriptions are included next.

- [ECS 45 1986]. This regulation states that the protocol consists of one cycle for each of the intervals \([\theta_y^n / 4, 2 \theta_y^n / 4], [2 \theta_y^n / 4, 2 \theta_y^n / 4], [3 \theta_y^n / 4, 3 \theta_y^n / 4] \) and \([\theta_y^n, \theta_y^n] \), then three cycles at the interval \([2 \theta_y^n, 2 \theta_y^n] \), and finally three cycles at each of the intervals \([2 (n + 1) \theta_y^n, 2 \theta_y^n] \) for \( n = 1, 2, \ldots \).
- [Castiglioni 2016]. The recommendations of [20, 35] do not account for the actual asymmetric rotation histories (i.e. not centered at zero rotation) when seismic shaking is combined with gravity loads; thus, a modification is proposed herein. The imposed displacement law is established in terms of the vertical displacement of the pushing point (Fig. 4); it consists of one cycle for each of the intervals \([d_y^n / 4, d_y^n / 4], [2 d_y^n / 4, 2 d_y^n / 4], [3 d_y^n / 4, 3 d_y^n / 4] \) and \([d_y^n, d_y^n] \), and two or three cycles at the intervals \([2 (n + 1) d_y^n + \Delta d_y^n, (2 + n) d_y^n + \Delta d_y^n] \) \( n = 0, 1, \ldots \). In these expressions, \( d_y^n \) and \( d_y^n \) are the yield displacements (corresponding to rotations \( \theta_y^n \) and \( \theta_y^n \), respectively), and \( \Delta d_y^n \) and \( \Delta d_y^n \) are the displacement amplitudes until the force-controlled part of the cycle reaches the force correspondent to gravitational load. For further clarity, Fig. 7 displays the proposed plastic cycles; positive forces induce hogging bending (downward force), \( F_y \) is the yielding force (corresponding to moment \( M_y \)), and \( F_y \) is the gravity force corresponding to the considered loading level.

Fig. 7 shows that both the positive (increasing downward displacement) and negative (decreasing downward displacement) branches consist of force-controlled and displacement-controlled segments; thus, the gravity force \( F_y \) is the border between them. The test end is defined with respect to \( F_y \); ordinarily, it arises in the positive branch: either in its force-controlled part (the specimen fails to develop gravity moments, Fig. 4); it consists of one cycle for each of the intervals \([d_y^n / 4, d_y^n / 4], [2 d_y^n / 4, 2 d_y^n / 4], [3 d_y^n / 4, 3 d_y^n / 4] \) and \([d_y^n, d_y^n] \), and two or three cycles at the intervals \([2 (n + 1) d_y^n + \Delta d_y^n, (2 + n) d_y^n + \Delta d_y^n] \) \( n = 0, 1, \ldots \). In these expressions, \( d_y^n \) and \( d_y^n \) are the yield displacements (corresponding to rotations \( \theta_y^n \) and \( \theta_y^n \), respectively), and \( \Delta d_y^n \) and \( \Delta d_y^n \) are the displacement amplitudes until the force-controlled part of the cycle reaches the force correspondent to gravitational load. For further clarity, Fig. 7 displays the proposed plastic cycles; positive forces induce hogging bending (downward force), \( F_y \) is the yielding force (corresponding to moment \( M_y \)), and \( F_y \) is the gravity force corresponding to the considered loading level.

Table 3 Parameters derived after the monotonic experiments (Table 2).
4. Results of the experiments

4.1. Monotonic experiments

This subsection describes and discusses the results of the monotonic tests (M⁺ and M⁻) that are listed in Table 2. Fig. 8 displays the measured moment-rotation curves. The plotted moments include the effect of the constant 5 kN load (Figs. 4 and 5).

The sudden drops in Fig. 8a–c (tests No. 1 and 2), Fig. 8e (tests No. 1 and 3) and Fig. 8g (tests No. 1 and 3) correspond to the brittle failure of the welding between the end-plate and the beam. In the other cases the tearing in the hooks and perforations (tabs) is observed. An example of each failure type is displayed in Fig. 9a and b, respectively.

The curves for each weld geometry that exhibit the same failure type; “Weld” refers to the rupture of the welding (break, Fig. 9a), and “Hook” means large deformation and slide of the hooks inside the upright perforations (tearing, Fig. 9b). The monotonic IU tests had not been carried out; the positive (downward) and negative (upward) values are taken from the FP-M⁺ and VS-M⁻ tests, respectively. Such correspondences have been established based on their behavior similarity. In both cases, slight corrections [5] are introduced, given that the material, yet being formally the same, corresponds to a different coil (Tables 1 and 2).

Fig. 8 and Table 3 show that the welding configuration significantly influences the connection monotonic performance, both in moment capacity and rotation ductility: as expected, the longer welding bead (FPIF), the better performance. More precisely, in all the VS tests, failure arises in the welding, while both FP and FPIF sagging tests always show a hook failure mode (tearing). Additionally, FP and FPIF hogging tests show both weld and hook failures. However, the influence of the welding path in both the initial and equivalent stiffness is not relevant, as the observed variations are rather erratic and are frequently uncorrelated with the alleged weld stiffness (this trend can also be seen in Fig. 10). Going to more particular issues, a moderate pinching-like phenomenon occurs in some tests for small pulling forces (No. 1 in Fig. 8b, No. 2 in Fig. 8d, No. 3 in Fig. 8f and No. 3 in Fig. 8h); it is apparently due to lateral slippage of the end-table hooks inside perforations of the upright. Finally, the difference between the positive and negative values of 0u can be explained by the early failure of the welding displayed in Fig. 8c.

4.2. Cyclic experiments

Analogously to Subsection 4.1, this subsection presents the results of the cyclic tests (C-0, C-ECCS, and C-CAS) listed in Table 2. Apart from the particular case of the FPIF-C-0 test, the limits of the cyclic experiments C-ECCS and C-CAS are selected from the positive and negative average values of the yield rotation in Table 3. Regarding this issue, the precise correspondences are as follows: the bounds of VS-C-ECCS, FP-C-ECCS, and FPIF C-ECCS are taken from VS, FP, and FPIF-A monotonic test, respectively; the bounds of IU-C-CAS and FPIF-C-CAS are taken from IU, and FPIF-B monotonic test, respectively. In this sense, Table 2 shows the right correspondence between the material coils and the matching monotonic and cyclic tests.

As discussed in Subsection 3.4, test FPIF-C-0 (Table 2) did not include the 5 kN load (Figs. 4 and 5); then, in order to investigate the influence of such force, Fig. 11 displays a comparison between the M-θ plots of test FPIF-C-0 and the first four cycles of test FPIF-C-CAS (for 50%). Fig. 11 shows that both proofs exhibit significant pinching, with important near-horizontal branches in the vicinity of the origin point. In the FPIF-C-CAS test, the bounds of the four plotted loops have been established as described in the previous paragraph. Conversely, in the FPIF-C-0 test, the bounds for negative rotation angle (sagging) have been extended by basically ignoring most of the aforementioned horizontal branches (pinching); otherwise, almost no energy dissipation would be observed in the left part of the hysteresis loop. This circumstance (the abnormal extension of the pinching branches in the sagging part of the FPIF-C-0 test) can be explained by the partial pullout of the hooks in the upright perforations (the safety rivet prevents the total pullout, Fig. 1a). These considerations endorse the use of the aforementioned 5 kN force in order to avoid those undesirable effects. However, it should be kept in mind that the hook sliding might feasibly occur in real unloaded rack segments undergoing seismic excitation.

Fig. 12 displays the moment-rotation loops of C-ECCS and C-CAS tests. Like in the monotonic experiments (Fig. 8), in Fig. 12 the positive
and negative moments and rotations correspond to downward and upward driving forces, respectively.

Analogously to Figs. 9, 13 displays representative examples of the failure modes. As in the monotonic experiments, the failure can be produced either due to weld break, or excessive hook/end-plate/upright deformation. Noticeably, the weld rupture always starts at the top front corner, as shown in Fig. 13 a.

Analogously to Tables 3, 4 displays the most meaningful output parameters of the cyclic tests (Table 2). In the column labeled “Date”, each test is identified by the date it was performed; “Mean” refers to the average of the experiments belonging to the same category. The column labeled “Gravity load percentage” contains the percentages of $F_y$ in the Castiglioni tests. In the subsequent columns, the maximum rotation ($\theta_{\text{max}}$) corresponds to the peak positive and negative values, the displacement ductility ($\mu$) refers to the considered average yield rotation (after the monotonic tests, Table 3), the number of cycles includes even the initial ones (although their encompassed area is rather small), the absorbed energy is the area encompassed by the hysteresis loops, the failing component is indicated as in Table 3 (for the monotonic tests), and, finally, the last column (labeled “Test end”) only refers to the Castiglioni experiments and describes whether the end is produced in the displacement or force-controlled branch.

Fig. 12 and Table 4 provide relevant information about the carried out cyclic experiments. For further clarity, Fig. 14 presents some of the results in Fig. 12, although organized differently.

The observations arising from Fig. 12, Table 4, and Fig. 14 are discussed next.

- **Common remarks.** All the tests exhibit a rather relevant ductility, both in terms of displacement and absorbed energy; even the first
cycles encompass a significant area. Also regarding ductility, failure arises after substantial stiffness and strength degradation. The most negative circumstance is the pinching effect due to the gap between the end-plate hooks and the upright perforations (Fig. 13b).

- **Comparison between ECCS and CAS.** As expected, in the C-ECCS tests, the cycles are asymmetric in terms of rotation; this can be explained by the difference in the positive and negative values of $\theta_y$ (Table 3). Conversely, the loops are more clearly asymmetric in the C-CAS tests; moreover, such asymmetry increases as the test progresses (Subsection 3.4). Global comparison between the ECCS and Castiglioni tests shows that, as expected, the C-CAS tests are more demanding than the C-ECCS ones, in the sense that the obtained performance of the tested connection (in terms of the number of cycles and dissipated energy) is smaller. This dissimilarity seems to indicate that the testing protocol in [2] should be generally preferred as being closer to reality (the actual behavior depends on the weight of the stored goods).

- **Comparison between CAS for 75%, 66%, 50% and 25% of $F_y$.** The results of C-CAS and C-ECCS tests tend to converge as the percentage of the gravity force decreases; this trend is predictable, as the assumed gravity force is the main difference between both testing protocols. Nonetheless, this tendency is interrupted when experiments of the same type present arbitrarily distinct failure modes; for example, for the FPIF test series (Fig. 12d), two of the specimens (50% and 60%, Fig. 14a) present a weld break while the other specimens (25% and 75%, Fig. 14b) do not.

- **Comparison between VS, FP, IU and FPIF.** As anticipated by the results of the monotonic experiments (Subsection 4.1), FPIF performs better than FP and, in turn, FP performs better than VS. This trend is clearly shown by the column “Failure mode” in Table 4: in all the VS and FP tests failure arises in the welding, while in the FPIF experiments such brittle failure mode occurs in two cases only. The
Castiglioni tests require particular discussion as they are the only ones including IU (Inverted U) welding; in this sense, as expected, the behavior of FPIF is better than that of IU even for weld break (CAS 50% and 66%, Fig. 14a). In the FPIF cases where weld failure is avoided, the behavior of such connection type is significantly better than IU (CAS 25% and 75%, Fig. 14b).

- **Particular comparison between FP-C-ECCS and FPIF-C-ECCS-I.** This contrast follows the general trends pointed out in the previous paragraph; more specifically, FPIF specimens do not present a weld failure, while FP ones do. On the other hand, as expected, the weld path does not considerably affect the cyclic behavior before such failure is produced. For higher clarity, plots FP-C-ECCS(1) and FPIF-C-ECCS-I are contrasted in Fig. 14c. FP-C-ECCS(1) has been chosen (instead of 2 or 3 tests) as it presents a later weld break and, consequently, clearly shows the aforementioned pre-failure plot coincidence.

- **Particular comparison between FPIF-C-ECCS and FPIF-C-ECCS-I.** This contrast is displayed in Fig. 14d. The difference between the negative angles of both tests can be easily explained by the definition of the cycle bounds. More significantly, larger sagging angles lead to smaller maximum hogging moments. This behavior is rather similar to the well-known Bauschinger effect (of materials behavior). It can be attributed to the higher deterioration (tearing) of the hook-perforation assemblies as it affects the connection moment capacity both for positive and negative moments. Comparison with FPIF-C-ECCS-II provides similar remarks.

As a supplementary remark, it is observed that the ECCS cyclic plots (Figs. 12a, c, e, 14c and d) show notable differences between the three specimens in terms of strength and stiffness degradation. These two issues are of paramount interest for understanding the seismic behavior of pallet racks’ beam-to-upright connections. Fig. 15a displays the averaged backbone curves for each weld configuration. The backbone curve can describe strength degradation; this graph is drawn by joining the maximum moment points ($M_i$) of each loading grade (group of three cycles with the same amplitude $\theta_i$). Fig. 15b displays, also for each weld configuration, plots of $S_i$ vs. $\theta_i$. Noticeably, in Fig. 15, FPIF-C-ECCS-I and FPIF-C-ECCS-II are not averaged with the
Table 4
Parameters derived after the cyclic experiments.

<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Gravity load percentage</th>
<th>$\theta_{\text{max}}$ (mrad)</th>
<th>$\mu$ ($\theta_{\text{max}}/\theta_y$)</th>
<th>Number of cycles</th>
<th>Absorbed energy (kJ)</th>
<th>Failing component</th>
<th>Test end</th>
</tr>
</thead>
<tbody>
<tr>
<td>VS-C-ECCS</td>
<td>28/11/2017</td>
<td>–</td>
<td>61 83</td>
<td>2.99 3.12</td>
<td>9</td>
<td>0.61</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>04/05/2018</td>
<td>–</td>
<td>61 83</td>
<td>2.99 3.12</td>
<td>9</td>
<td>0.57</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>07/05/2018</td>
<td>–</td>
<td>61 83</td>
<td>2.99 3.12</td>
<td>10</td>
<td>0.66</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td>Mean</td>
<td>10/01/2018</td>
<td>–</td>
<td>61 83</td>
<td>2.99 3.12</td>
<td>9</td>
<td>0.57</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td>IU-C-CAS</td>
<td>11/01/2018</td>
<td>–</td>
<td>82 133</td>
<td>4.00 3.99</td>
<td>11</td>
<td>1.37</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td>VS-C-ECCS</td>
<td>22/05/2019</td>
<td>75%</td>
<td>89 137</td>
<td>4.33 4.11</td>
<td>12$^{1/3}$</td>
<td>1.70</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>IU-C-CAS</td>
<td>22/05/2019</td>
<td>50%</td>
<td>121 53</td>
<td>7.20 1.64</td>
<td>8</td>
<td>0.56</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>22/05/2019</td>
<td>25%</td>
<td>114 54</td>
<td>6.79 1.67</td>
<td>10</td>
<td>0.76</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>FPIF-C-ECCS</td>
<td>05/05/2018</td>
<td>–</td>
<td>92.2 41</td>
<td>5.49 1.27</td>
<td>7</td>
<td>0.46</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>05/06/2018</td>
<td>–</td>
<td>91 232</td>
<td>7.78 7.95</td>
<td>24</td>
<td>3.99</td>
<td>Hook</td>
<td>–</td>
</tr>
<tr>
<td>FPIF-C-ECCS-</td>
<td>03/05/2018</td>
<td>–</td>
<td>89 230</td>
<td>7.61 7.86</td>
<td>23</td>
<td>3.92</td>
<td>Hook</td>
<td>–</td>
</tr>
<tr>
<td>FPIF-C-ECCS-</td>
<td>21/11/2017</td>
<td>75%</td>
<td>75 146</td>
<td>6.41 5.00</td>
<td>16</td>
<td>2.87</td>
<td>Hook</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>15/06/2020</td>
<td>–</td>
<td>200 28.9</td>
<td>9.57 0.99</td>
<td>8$^{1/3}$</td>
<td>1.46</td>
<td>Hook</td>
<td>–</td>
</tr>
<tr>
<td>FPIF-C-CAS</td>
<td>27/07/2018</td>
<td>66%</td>
<td>120 32</td>
<td>5.74 1.09</td>
<td>6$^{1/3}$</td>
<td>0.51</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>27/07/2018</td>
<td>50%</td>
<td>133 32</td>
<td>6.36 1.09</td>
<td>7$^{1/3}$</td>
<td>0.61</td>
<td>Weld</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>16/06/2020</td>
<td>25%</td>
<td>256 29.3</td>
<td>12.25 1.00</td>
<td>11$^{1/3}$</td>
<td>1.46</td>
<td>Hook</td>
<td>–</td>
</tr>
</tbody>
</table>

* The ductility is determined from the actual yield angle ($\theta_y$).
rest of FPIF-A specimens for presenting different cyclic bounds. Fig. 15 shows that strength and stiffness degradations occur more progressively for FPIF than for the rest of the specimens due to the hook ductile failure mode; the failure of FPIF specimens (Table 4) is smoother than the sudden weld break of the rest of the welding configurations. Furthermore, both strength and stiffness degradations prove to be more severe in the hogging branch, especially for FPIF. Finally, Fig. 15(b) shows that FPIF exhibits a significantly higher value of initial stiffness, as already observed in the monotonic tests (Table 3); nonetheless, as the test progresses, this difference vanishes.

Table 4 displays the dissipated energy in the cyclic experiments; as discussed, the values for FPIF are significantly higher than those for the other welding paths (VS, FP, and IU). However, in order to highlight the seismic capacity of the proposed welding solution, these values need to be compared with common input and hysteretic energy levels in medium-to-high seismicity regions; the following paragraph contains a preliminary appraisal.

Nowadays, Japan is the only country that includes input energy spectra in its regulations; the Japanese design code [40] states that the maximum input energy (i.e. corresponding to the spectrum plateau and to the most seismic prone zones of Japan) in terms of equivalent velocity ranges between $V_E = 234$ cm/s for rock and $V_E = 312$ cm/s for soft soil. These values can be converted into actual input energy as $E_I = \frac{1}{2} m (V_E)^2$; in that expression, $m$ is the mass of the structure under consideration. On the other hand, studies carried out from Colombian and Turkish records [41, 42] provide rather similar results. All this information is compared next with the energy dissipation capacity of racks. A typical value of the beam span length for racks is 2 m, and the maximum load can be assumed as 3 kN/m. As the rack lateral motion during severe seismic shaking is basically rigid-body (i.e. rigid uprights and beams connected by rotational springs), it can be reasonably assumed that the seismic input energy is distributed rather uniformly between all the beam-to-column connections. Therefore, it can be concluded that a mass of 300 kg corresponds to each connection; in that situation, the input energy per connection (in soft soil) is $E_I = \frac{1}{2} m (V_E)^2 = \frac{1}{2} 300 \times 3.12^2 = 1460$ J $= 1.46$ kJ. Table 4 shows that this value is clearly below the
energy absorption capacity of the tested FPIF connections (ranging between 2.87 kJ and 3.99 kJ, except for CAS loading protocols, which are more demanding); conversely, it exceeds the capacities for VS, IU, and FP. Moreover, the seismic hysteretic energy (i.e. the energy contributed to damage) will be smaller than the input one. These considerations seem to indicate the seismic ability of the proposed welding solution.

4.3. Comparison between monotonic and cyclic experiments

This subsection contrasts the cyclic and monotonic proofs. In this sense, Fig. 16 displays comparisons between the cyclic tests (Fig. 12) and the corresponding monotonic experiments (Fig. 8). The cyclic and monotonic plots are drawn in grey and black, respectively; this criterion is also considered for the additional information on the failure type (hook or weld). Fig. 16a and b refer to VS and FP specimens, respectively; given that the Castigliani tests for such weld configurations were not performed (Fig. 12b and d), all the plotted curves correspond to the ECCS protocol (this is indicated in the captions). Fig. 16c and d relate to FPIF specimens for ECCS and CAS testing protocols, respectively. In order to emphasize the suitability of the comparisons in Fig. 16, it should be kept in mind that tests FPIF-A-M³, FPIF-A-M⁵ and FPIF-C-ECCS (Fig. 16c) use the same steel coils (1–3–4, Table 2); also tests FPIF-B-M³, FPIF-B-M⁵ and FPIF-C-CAS (Fig. 16d) utilize the same identical coils (2–3–5, Table 2).

Fig. 16 shows that, as expected, the monotonic plots are basically the backbone curves of the cyclic loops. More precisely, the closest coincidence emerges in the initial cycles vs. the growing segments of the monotonic curves; conversely, the differences between both types of plots become more significant as damage progresses. Deeper considerations are discussed next, separately for the ECCS and CAS experiments.

- **ECCS tests.** Fig. 16a–c show that the force (moment) strength is higher in the monotonic tests (for both hogging and sagging); this circumstance can be explained by the cumulated damage in the hook-perforation assembly and the low-cycle (plastic) fatigue in the weld.

- **CAS tests.** Fig. 16d shows a meaningful coincidence between the hogging monotonic and cyclic branches corresponding to the same failure type. This proximity might be explained by the smaller absorbed energy, thus leading to less cumulated damage (Table 4). Further research is necessary to confirm this trend.

In brief, Fig. 16 shows that, except perhaps for the CAS tests, the monotonic laws cannot be considered, in their present form, for numerical dynamic analysis because they differ significantly from the cyclic laws.

5. Conclusions

This paper aims to investigate the seismic performance of racking systems; in order to do this, vertical cantilever monotonic and cyclic tests of speed-lock upright-to-beam connections are presented and discussed. Such connections consist in welding the beam to an intermediate L-shaped end-plate; in its turn, this element is connected to the upright through a hooked assembly. Noticeably, the beam section is made of two nested C-lipped profiles. The tested specimens differ in the weld path between the end-plate and the upright. Four beads geometries have been utilized; in crescent order of weld bead length, they are: VS (Vertical Sides only), IU (Inverted U, i.e. vertical sides and top flange), FP (Full Perimeter, i.e. vertical sides and top and bottom flanges) and FPIF (Full Perimeter and Internal Flanges, i.e. FP but welding not only the external flanges of both nested C-lipped profiles, but also the internal ones). This last weld configuration constitutes the main contribution of this work and aims to guarantee that the failure arises in the hooked assembly (connection between the end-plate and the upright) instead of in the weld; this shift is expected to generate a rather ductile failure mode. The cyclic tests have been conducted according to both the European
document [35] and a recently proposed modification (CAS, [2]); its main novelty is a modification of the imposed displacement protocol accounting for the gravity loads influence. In both testing protocols, the test arrangement of [20] (even the 5kN load) has been used as it is more suitable for pallet racks.

The carried-out experiments provide the following major conclusions:

- The four tested weld paths provide different results; in global terms, as expected, the longer the weld beads, the better the performance. This trend is observed in terms of strength, stiffness, and absorbed energy; regarding this last issue, shorter beads generally lead to early weld break, while this undesired brittle failure mode is totally avoided in the FPIF weld configuration tested under the ECCS protocol. Preliminarily, this indicates a satisfactory behavior of the proposed weld bead geometric configuration.
- The CAS testing protocol is more demanding than the ECCS one and fits better the actual conditions of racks under the combined effect of seismic excitations and gravity loads.
- The cyclic test results are very sensitive to the testing protocols (ECCS vs. CAS) and also little sensitive to the cyclic bounds. Further research might be necessary to define the protocol and the limits better.
- Although the monotonic plots are basically the backbone curves of the cyclic loops, their comparison shows significant differences in their final segments, while the pre-failure ones are rather similar. This trend can be explained by the influence of the cumulated damage in the cyclic tests.

Further research, currently in progress, involves numerical simulations of these experiments, and also additional experimental & numerical studies, aiming to furtherly understand the influence of the proposed FPIF weld solution and to obtain behavior factors of full racks.

**List of acronyms**

- **C** Cyclic.
- **CAS** [Castiglioni et al. 2016].
- **ECCS** [ECCS 451986]. I and II refer to the cycles limits (Table 2), 0 means that the 5 kN force (Fig. 5) was not used. The alike tests are distinguished by adding (1), (2), (3).
- **EN** [EN 16681 2016].
- **FP** Full Perimeter.
- **FPIF** Full Perimeter and Internal Flanges.
- **IU** Inverted U.
- **VS** Vertical Sides.
- **M** Monotonic (+/- downward/upward). A and B refer to the employed steel (Coil No., Table 2).

**List of symbols**

**Main symbols**

- \( \Delta \) Increment
- \( \nu \) Poisson ratio
- \( \theta \) Rotation angle (Fig. 4b, 6)

**Indexes**

- \( d \) Design
- \( EN \) Refers to the employed EN (Euro Norm) (Fig. 6)
- \( g \) Gravity (Fig. 7)
- \( i \) Index.
- \( \max \) Maximum value
- \( R \) Resisting
- \( s \) Steel
- \( u \) Ultimate (Fig. 6)
- \( y \) Yielding (Fig. 6)
- \( 0 \) Initial (Fig. 6)
- \(+, –\) Downward, upward displacement (hoggging, sagging, respectively)

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**Declaration of Competing Interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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**References**
