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Stiffness and resistance of brace-to-upright joints with lipped channel braces assembled flange-to-flange

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ABSTRACT

Industrial cold-formed steel pallet racks are constructed of upright frames bolted to braces in the cross-aisle direction. The flexibility and resistance of these brace-to-upright joints play a key role in the structural behavior of pallet racks in the cross-aisle direction. In this paper, the behavior of these joints has been evaluated through monotonic and cyclic experimental tests to determine the dominant failure modes and the stiffness. The resultant cyclic structural response is pinched, which suggests that upright frame connections are non-dissipative. An analytical model based on the Component Method has been developed to evaluate the ultimate resistance and the flexibility of these bolted connections, with braces oriented flange-to-flange. The model is capable of identifying the influence of each component on the connection structural response. A comparison with results obtained by a Finite Element numerical model, typically used in common practice, in which members are modelled by elastic frame elements and connections are assumed rigid is conducted. The numerical model overestimates the effective stiffness by nearly 14 times. The same model, when modified with a linear link with a force-displacement response determined from the analytical model, provides reliable results in agreement with the experiments. The developed modelling approach is recommended for adoption by practitioners for the design of steel storage pallet racks.

1. Introduction

Industrial cold-formed steel (CFS) pallet racks consist of upright frames and pallet beams. Upright frames are formed by uprights connected to braces which provide stiffness in cross-aisle direction Fig. 1a). Uprights are open cold-formed thin-walled steel profiles and braces are usually channel sections. In the down-aisle direction Fig. 1b), the stability of racks relies on the beam to upright connections and the nonlinear structural behavior of these connections and its effect on the structural response of Moment Resisting Frames is well studied [1–9]. However, little work exists on the stiffness of steel storage rack upright frames with bolted connections in the cross-aisle direction. Experimentally, there is some guidance provided by relevant codes and specifications. The rack codes BS EN 15512 [10] and European Standard (FEM-2008) [11] detail the experimental protocol used to determine the shear stiffness of upright frames. To consider both bending and shear effects, Gilbert and Rasmussen [12] proposed an alternative test setup to that suggested by Australian Standard AS4084 [13]. The authors highlighted that the local flexibility of the joints plays a key role in the

performance of the frame. Sajja et al. [14] conducted numerical and experimental investigations on the shear stiffness of rack upright frames using various upright sizes, restraints, and bracing configurations. In [15], an analysis of welded rack braced frames made using a rigid joint model demonstrated a stiffness several times greater than the stiffness of the actual frame.

Far et al. [16] have improved upon an FE model of the upright frame by reducing the axial stiffness of the braces, which effectively reproduces the effect of joint flexibility on the system. They concluded that Timoshenko and Gere's theory [17] overestimates the shear stiffness by nearly a factor of 10, and linear numerical models are not able to match the experimental results.

The same conclusions were highlighted by Rao et al. [18] comparing the difference in the shear stiffness evaluation by adopting theoretical formula proposed by RMI code [19] and experimentally determined according to FEM code [20].

Thombare et al. [21] performed a nonlinear buckling analysis of CFS storage rack frames, quantifying the influence of different diagonal brace configurations. Godley and Beale [22] investigated the effects of

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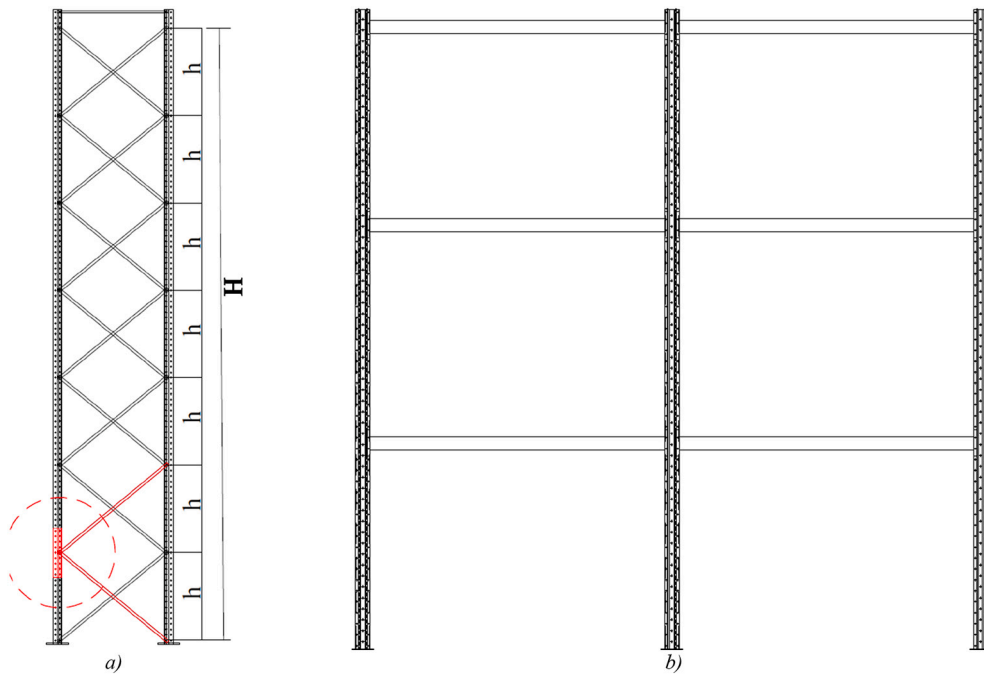


Fig. 1. Steel storage selective pallet rack structural schemes and brace-to-upright connection in red. a) Cross-aisle direction – Bracing system b) Long-aisle direction – Moment resisting frame. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

looseness of connection cyclic structural response on the ultimate load-carrying capacity of pallet rack frames along the cross-aisle direction.

In [23] the connection response was investigated via an experimental setup developed for this purpose. In general, any prior consideration of the complete system behavior of cross-aisle frames has required experimental testing to determine the shear stiffness of braced frames, as it is dependent on local connection flexibility.

Talebian et al., [24,25] developed a FE numerical model capable of accurately reproducing the transverse shear stiffness of upright frames using shell elements. The numerical model was adopted to highlight and discuss the factors contributing to the transverse shear deformation of bracing systems with both lip-to-lip and back-to-back diagonal braces.

Furthermore, while the seismic system response of steel storage pallet racks in cross-aisle direction has been investigated in [26,27] with pushover and dynamic analyses, the local cyclic response of the brace-to-upright joints remains under-studied, and is a focus of the present paper, in addition to the proposal of an analytical method capable to estimate the connection structural response in terms of strength and stiffness.

Experimental testing remains a reliable method for determining the flexibility and resistance of the connections of braced cold-formed thin-walled frames used in storage racks, but it can be time-consuming and expensive. This motivates the development of an analytical method. In addition, the paucity of a reliable modelling approach that captures local effects, which could be adopted in common practice further motivates the work.

In the initial part of the paper, laboratory experiments are conducted under monotonic and cyclic loading on bolted connections with flange-to-flange braces. The paper continues by comparing the accuracy of different analysis methods with test results and proposes a simple analytical approach to approximate the connection flexibility and its resistance. The proposed method, based on the general Component Method [28–32], leads to the development of a linear force-displacement curve, which can be incorporated in an elastic analysis to better obtain values for the stiffness of the entire frame. The proposed approach can be used to evaluate the failure mode of the joint and describe its structural performance.

2. Experimental campaign

The connections studied herein join upright frames to diagonal braces. These braces have a lipped C cross-section shape with 25 mm flanges, a 25 mm web, and 8 mm lips, with a thickness of 1.5 mm, while the uprights have a complex thin-walled open cross-section shown in Fig. 2. Steel properties were obtained by tensile tests on three coupons extracted from the steel coil used for each type of member – brace or upright. In particular, the steel coils belong to two different manufactured batches: braces (S250GD) and uprights (S350GD). Mean values of mechanical properties are listed in Table 1, where f_{ym} and f_{um} are the average yield and ultimate strengths, respectively, of the three coupon tests, for each member type (Fig. 3). Coupon tests are conducted under monotonic displacement control at a rate of 2 mm/min. Although this value is relatively high for a perfect evaluation of the yielding stress and the stress relaxation is neglected [33], the chosen strain ratio is in accordance with [34], and could be considered an acceptable testing speed in the industry, which requires a much shorter testing time. Moreover, it is the same used in monotonic full-scale tests devoted to assessing the structural response of rack connections, and therefore it could be assumed to be adequate for engineering purposes.

In frames used in industrial storage racks, braces can be oriented and bolted either back-to-back or front-to-front (lip to lip) as shown in Fig. 4a and b, respectively. Results reported in [16] indicated that the braced frames made of lipped C channels assembled back-to-back have a higher stiffness than those assembled lip-to-lip. In C channels assembled lip-to-lip, loads are applied to the bolt ends and cause the torsion of the upright with larger local deformation.

To increase the bearing resistance of the braces while slightly reducing the stiffness of the connection, braces are placed in the configuration shown in Fig. 4c, in which two flanges per member are fixed (referred to as flange-to-flange). In the proposed configuration, each brace has two shear planes. The load transferred by the brace acts at the center of the bolt, the location of maximum bending moment, and near the ends of the bolt shafts, close to the upright flanges. This mechanism results in the bearing of the connected parts. Bolts are not pre-tensioned and bolt tightness does not affect the connection force transfer mechanism between diagonals and uprights.

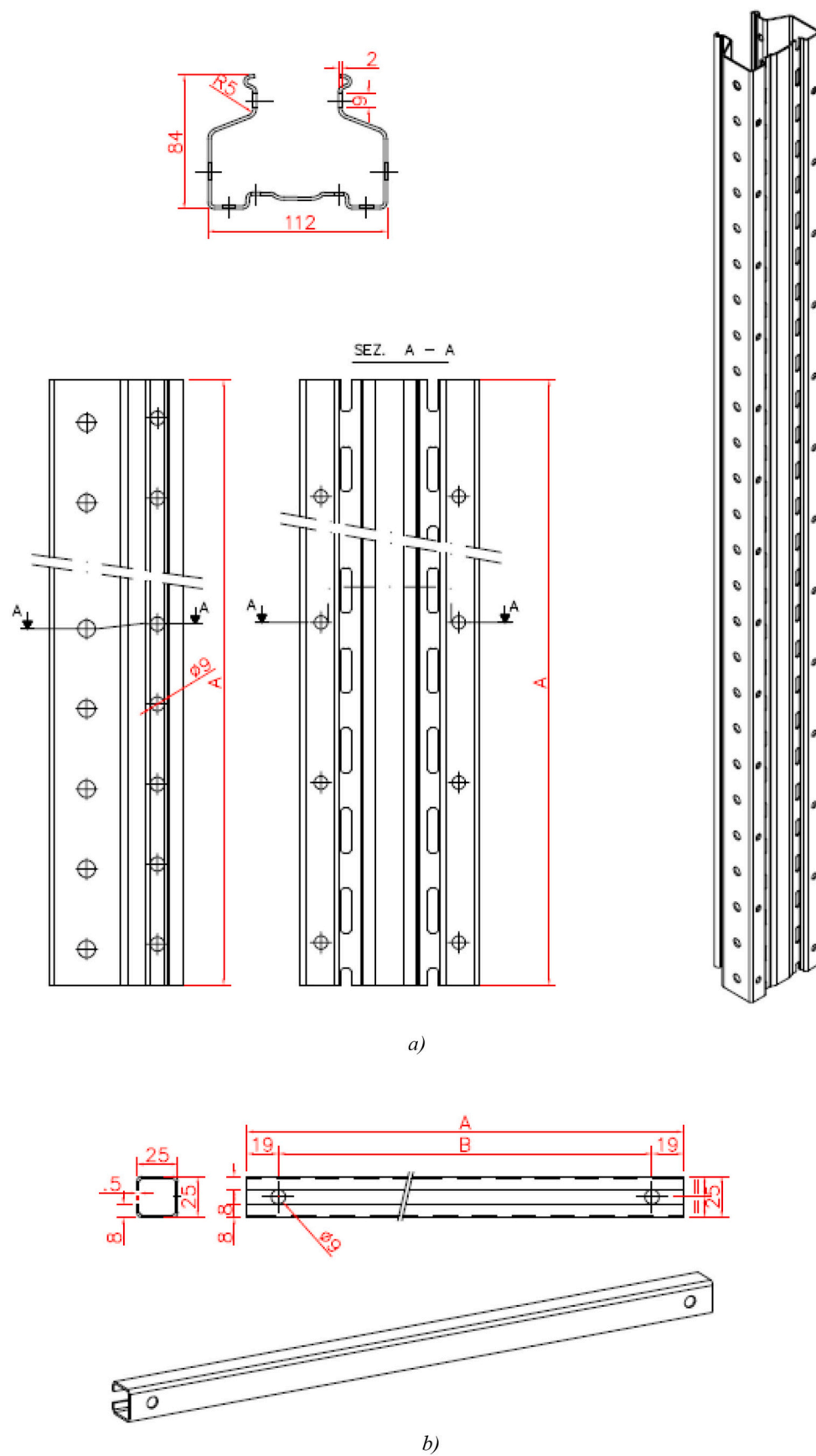


Fig. 2. The geometry of tested members. a) upright; b) brace.

Table 1
Mechanical features of tested members.

Member	f_{ym} [N/mm ²]	f_{um} [N/mm ²]	Steel Grade
Upright	432	465	S350GD
Braces	344	416	S250GD

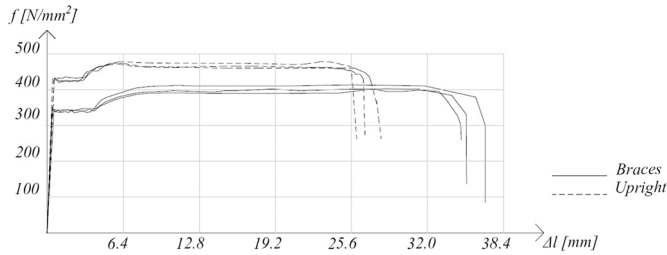


Fig. 3. Stress-elongation curve obtained through three coupon tests on the coil used for each member (braces and upright).

3. Experimental setup

The upright frame connection shown in Fig. 5 was tested to establish its stiffness and strength. The specimen was fabricated by cutting a brace-to-upright joint from a frame (Fig. 1). The specimens are made up of a segment of the upright, one short brace, and one long brace. The test setup, shown in Fig. 5, follows the experimental procedure developed in [23].

The goal of the experimental campaign is to induce statically determinant axial forces into the specimens. To achieve this, boundary conditions between specimen and test frame were carefully considered. For all specimens, two base plates are attached to both ends of the upright. The bases have a bolt M10 class 8.8 [31], aligned with the center of gravity of the section (Fig. 5 (a)). The upright end section is pinned, as shown in Fig. 5 (a). The other end can slide due to slotted holes in the plate (Fig. 5 (b)). Both upright ends are torsionally restrained. One M8 class 8.8 connects the braces and uprights, forming a pin (Fig. 5 (c)). The braces are placed flange-to-flange. The connections between the braces and the testing frame are designed as overstrength to prevent failure.

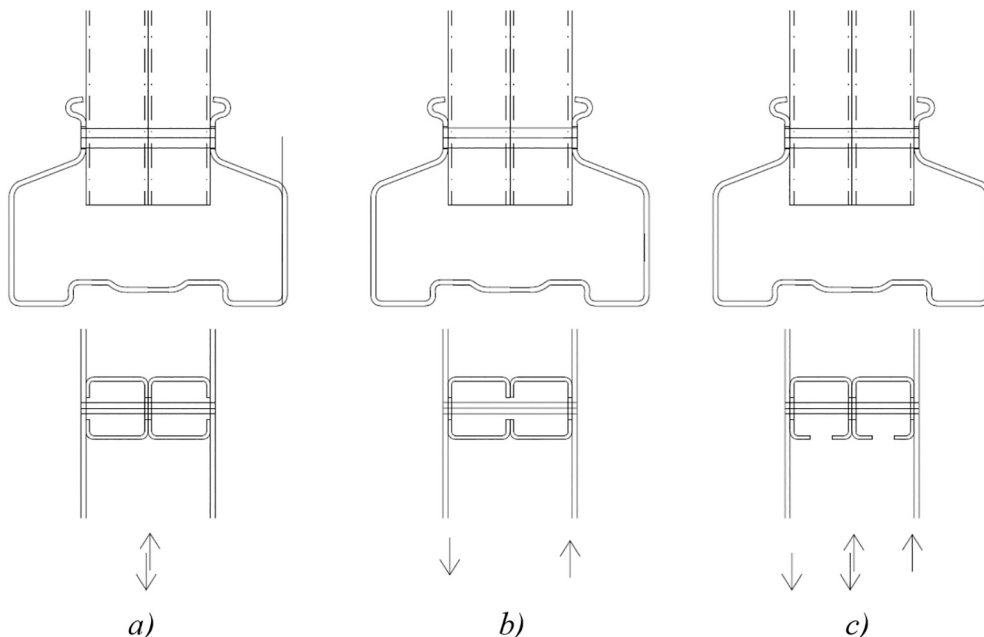


Fig. 4. Typical configurations of the upright frame bracing members (a) back-to-back (b) front-to-front (c) proposed flange-to-flange configuration. Plane view.

The connection between the short brace and the testing frame is hinged and is formed by three M8 class 8.8 bolts and one M16 class 8.8 bolt (Fig. 5 (d)). The long brace is connected to the testing machine with three M8 class 8.8 bolts and one weld (Fig. 5 (e)).

In Fig. 6, restrained displacements and rotations are identified for all nodes where external constraints are placed. In all tests, the long brace is kept vertical. The column is mounted diagonally. The load is applied to the long brace end (the control point) by a hydraulic actuator through a spherical bearing. In monotonic tests, the load is applied upwards to induce tensile axial force in the long brace, compressive axial force in the short brace, and tensile axial force in the lower segment of the upright. The force applied to the brace is measured using a force transducer. The vertical displacement is measured at 205 mm (s1) and 47 mm (s2) from the axis of application of the external load (Fig. 6). The load is gradually increased until the connection reaches the failure load. The member strains are measured by strain gauges installed at different positions (Ω_i in Fig. 6).

4. Monotonic tests

The monotonic tests are summarized in Table 2. Tests were performed under displacement control with a load rate of 2 mm/min. The non-dimensional monotonic force-displacement of the control point curve is shown in Fig. 7 for all specimens. Specimens are identified with the symbol “mi_{Mj}” where, “m” means monotonic test, “i” identifies the number of the test and “Mj” specifies that a bolt with a diameter of “j” mm is adopted to connect the braces and the upright. Test m1_{M10} differs from the others in that an M10 bolt was used to join the upright with the braces instead of an M8 bolt.

In Fig. 7 all non-dimensional monotonic force-displacement curves are shown. The experimental results are made dimensionless with respect to those of test m1_{M8}. Non-dimensional force-displacement curves are obtained dividing data for the maximum load and ultimate displacement of test m1_{M8}, owing to the impossibility of presenting absolute data for commercial products. The non-dimensional maximum force P_{exp} , the non-dimensional initial elastic stiffness K_{exp} with respect to that of test m1_{M8} and the failure mode of the tested joints are summarized in Table 3. The connection initial elastic stiffness is the slope of the straight line for the linear portion of each experimental curve, where according to the 1/10th slope method, the straight line intersects the

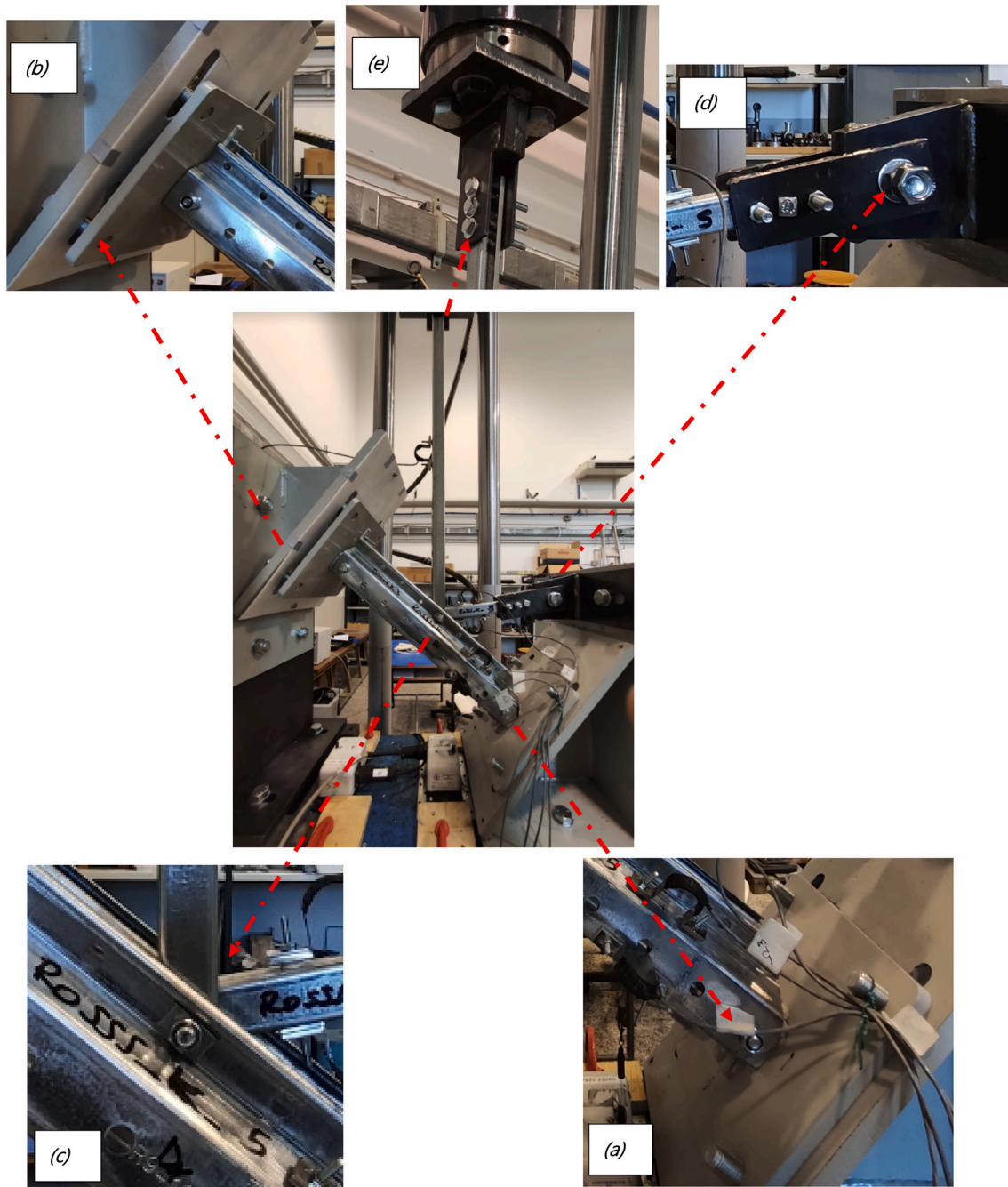


Fig. 5. Setup.

tangent at the origin and the tangent that has a slope of one-tenth the tangent at the origin, neglecting branches affected by slippage [35].

Observed cases of failure were consistently the bending of the bolts followed by the bearing of braces Fig. 8. Only in test m1_{M10}, where the diameter of the bolt is increased, from 8 mm to 10 mm, the weakest component is given by the long brace and the failure was due to the tearing of the bolt hole of the braces, as depicted in Fig. 9. Only by increasing the diameter of the bolt, the maximum non-dimensional load increases to $P_{exp} = 1.15$, and the elastic stiffness to $K_{exp} = 1.23$ increase (Table 3).

The permissible sway of rack systems is $v_{max} = H/200$ [15], which for a medium high rack system $H = [4 \text{ m} \div 6 \text{ m}]$, corresponds to $v_{max} = [20 \text{ mm} \div 30 \text{ mm}]$. In all specimens the failure occurs at a displacement at the

control point (at the end of the long brace) greater than 15 mm ($d \approx 0.63$, see Fig. 7, test m1_{M10}), which corresponds to a horizontal displacement of one braced bay $v_i = d/\cos(\alpha) \approx 21 \text{ mm}$, with α the angle between the horizontal axis and the brace axis (Fig. 10).

The horizontal displacement of only one braced bay v_i is already greater than the maximum displacement v_{max} of the whole rack system for $H = 4000 \text{ mm}$ and slightly lower than v_{max} for a 6 m high rack (Fig. 10). It can be concluded that within the service limit displacement, the post elastic behavior of the connection can be neglected, and the structural response of the joint can be approximated by a linear elastic link with stiffness K_{exp} .

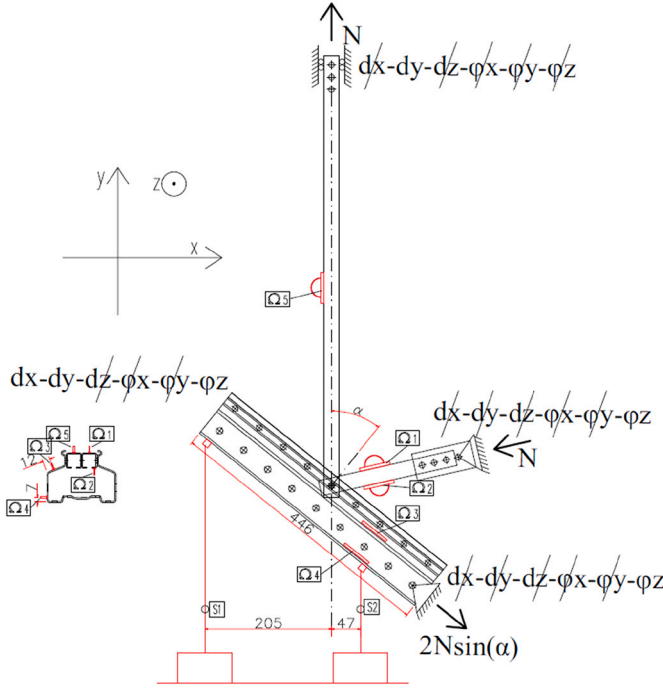


Fig. 6. Instrumentation schematic and restrained degrees of freedom.

Table 2
Monotonic tests with a long brace in tension.

Test	Upright	Brace	Bolt	Test
m1 _{M8}	110	25 × 25 × 8	M8	Monotonic – long vertical brace in tension
m2 _{M8}	110	25 × 25 × 8	M8	Monotonic – long vertical brace in tension
m3 _{M8}	110	25 × 25 × 8	M8	Monotonic – long vertical brace in tension
m1 _{M10}	110	25 × 25 × 8	M10	Monotonic – long vertical brace in tension

mi_{Mj}: m = monotonic test; i = number of the test; Mj = bolt diameter.

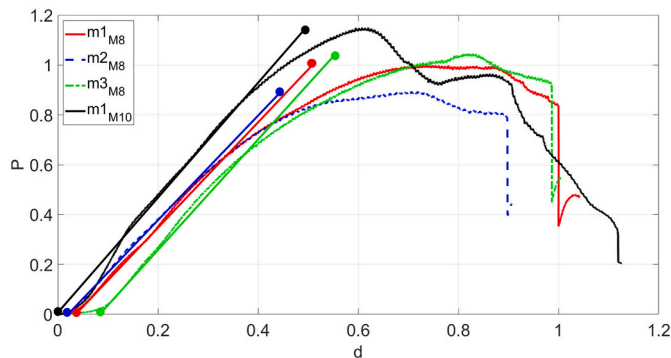


Fig. 7. Experimental non-dimensional monotonic force-displacement curve and linear elastic curves identified by double circles for all tested specimens.

5. Cyclic tests

Cyclic loading above the endurance limit inflicts measurable permanent damage in the specimen (cumulative damage), therefore, the strength and deformation capacities of the specimen undergo degradation as the number and amplitude of cycles increase. Cyclic tests were conducted to assess the structural response of members under seismic

Table 3

Non-dimensional maximum load P_{exp} , non-dimensional initial elastic stiffness K_{exp} and failure mode for all specimens.

Test	P_{exp}	K_{exp}	Failure mode
m1 _{M8}	1.00	1.00	Bolt bending and bearing of brace
m2 _{M8}	0.89	0.98	Bolt bending and bearing of brace
m3 _{M8}	1.05	0.99	Bolt bending and bearing of brace
m1 _{M10}	1.15	1.23	Bearing of brace

actions. The cyclic testing program is summarized in Table 4. The cyclic force-displacement response of the specimens is illustrated in Figs. 11 and 12. Specimens are identified with the symbol “c_{Mj-dhz}” where, “c” means cyclic test, “i” identifies the number of the test, “Mj” specifies that a bolt with a diameter of “j” mm is adopted to connect the braces and the upright and “dhz” identifies the displacement history ($z = A$ for displacement history A; $z = B$ for displacement history B). The specimens are nominally identical to the M8 bolted specimens in the monotonic test program, to enable comparisons across the load protocols.

To assess the extent of degradation, two variable quasi-static displacement histories (d_h) were applied (Table 4). The first displacement history (d_{hA} , Fig. 11) has a pattern similar to that proposed in [36]. The initial displacement was assumed equal to $d_0 = 10$ mm, the displacement corresponding to the yielding of the system, highlighted in the variation in the slope of the initial elastic load-displacement curve (one-tenth the tangent at the origin). It is followed by groups of three cycles whose amplitude increases until failure (Fig. 11). The second displacement history (d_{hB} , Fig. 12) is recommended by ECCS [37]. In this protocol, the displacement history begins with elastic cycles followed by groups of three cycles whose amplitude increases until failure (Fig. 12). These displacement scenarios are intended to simulate the loading experience by a connection during a seismic event [36]. Investigated bracing systems have a symmetric structural response in which diagonal braces work both in tension and compression and are designed not to buckle under seismic action. The effective behavior at the joint has been reproduced preventing the buckling of diagonals by applying a tension force at the long diagonal connected to the testing machine and adopting a compressed short diagonal. The set-up achieves the ultimate resistance of the connection.

The cyclic non-dimensional force-displacement curves are shown in Fig. 13 for all specimens. The non-dimensional maximum load P_{exp} and failure mode recorded in cyclic tests are listed in Table 5. The same failure mode (the bearing of the brace with the bolt bending) obtained at the same imposed displacement and a maximum ratio of 1.04 between the maximum load achieved in test with a different displacement history, highlight the connection structural response to be not affected by the displacement scenarios.

In Fig. 14 the average monotonic (m_{M8}) and cyclic curves (c_{M8}) for connections with M8 bolt are shown. It should be noted that the backbone cyclic curve aligns well with the monotonic curve, as highlighted by the maximum ratios between the average monotonic (m_{M8}) and backbone (back) values for the initial elastic stiffness (K) maximum load (P) and ultimate displacement (d_u), $K_{mM8}/K_{back} = 1.03$, $P_{mM8}/P_{back} = 1.06$ and $d_{u,mM8}/d_{u,back} = 1.00$, respectively ($1.0 =$ perfect estimation); as expected, the monotonic curve is slightly stronger because damage is not included.

The deformations of specimens observed during cyclic tests are shown in Fig. 15. Like in the monotonic tests, a translational softness due to bolt bending and bearing between bolts and the uprights is observed.

6. Energy dissipation characterization

Fig. 16 shows the dissipated energy versus the number of cycles for each specimen. The dissipated energy at the z -th cycle is expressed as: $\sum_{i=1}^z E_i$, where E_i is the area inside the i -th hysteresis loop of the force-

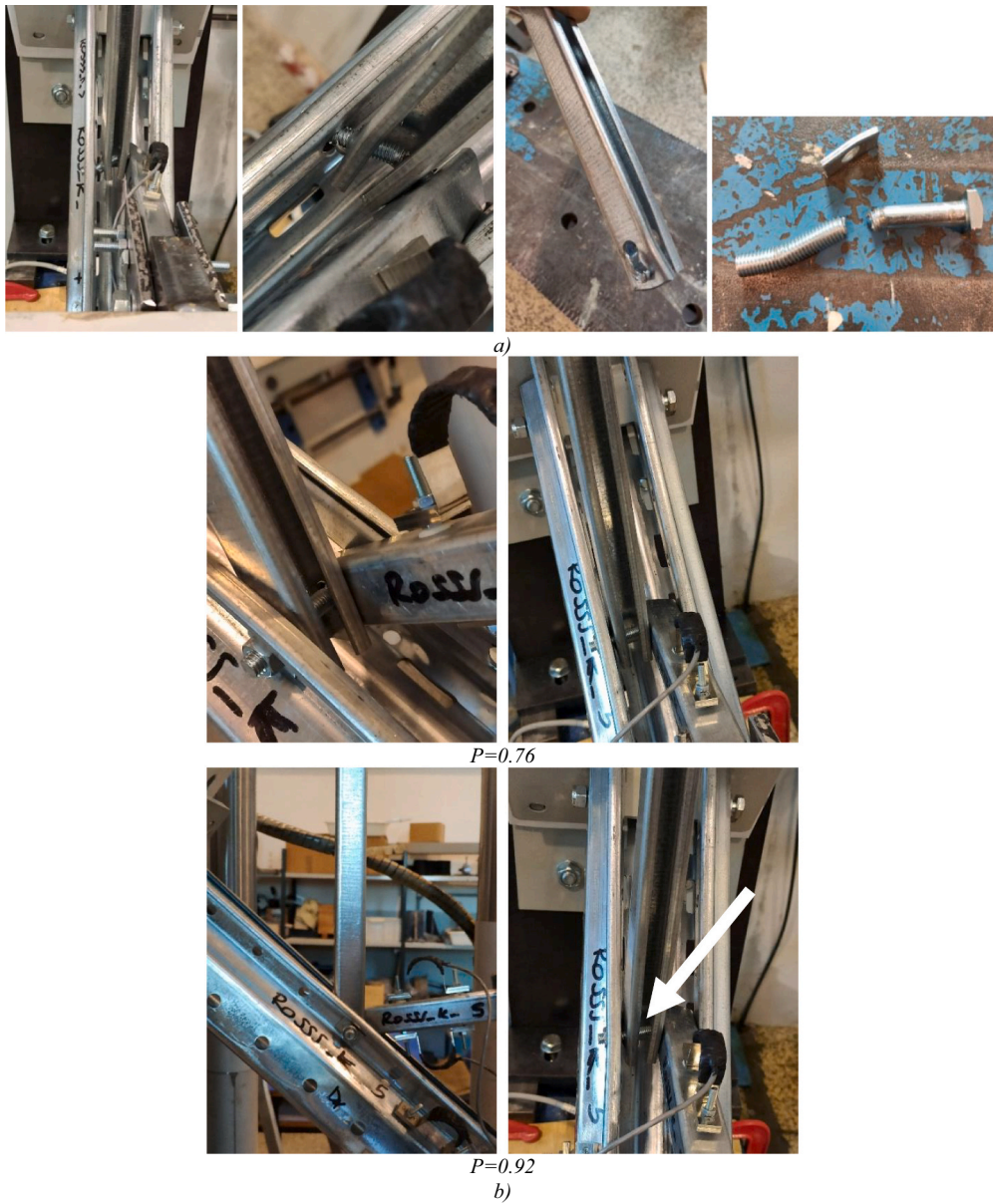


Fig. 8. a) Test $m1_{M8}$ – Specimen after the collapse. b) Test $m3_{M8}$ – Specimen deformation for different load steps and identification of the bending of the bolt (white arrow).

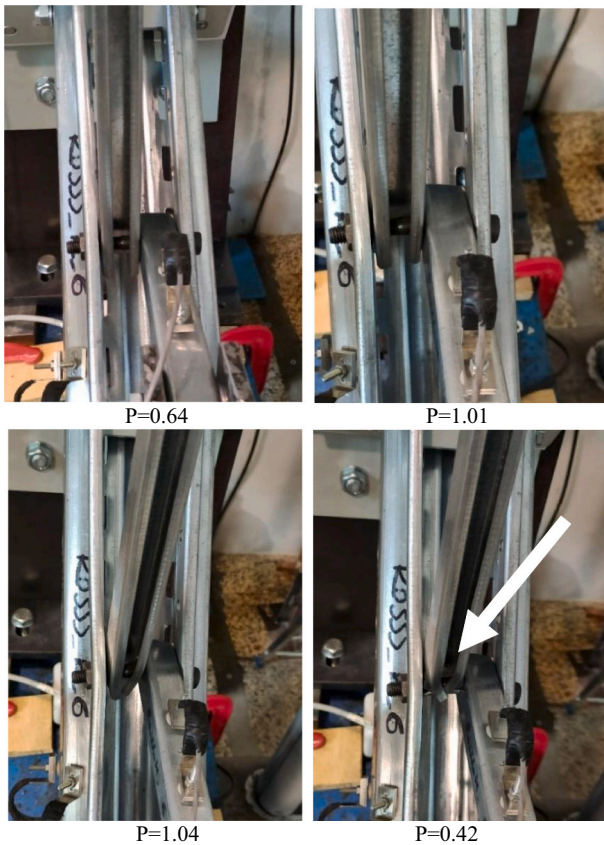


Fig. 9. Test m1_{M10} - Specimen deformation for different load steps and identification of the tearing of bolt hole of the braces (white arrow).

displacement curve.

Because of the pinched hysteretic response observed in the experimental testing, hysteresis loops are unstable. This further manifests as a reduction in dissipated energy in cycles following the first cycle of the three-cycle groups. This is shown in Fig. 17 in which the dissipated energy per cycle, normalized to the dissipated energy in the first cycle of three-cycle groups is presented (e.g. E_1/E_1 , E_2/E_1 and E_3/E_1 for the first three-cycle group, E_4/E_4 , E_5/E_4 and E_6/E_4 for the second three-cycle group, where the subscript corresponds to the cycle number, etc.).

The dissipation of energy in the last cycle is similar across all tested connections and is $\approx 6.5 \times 10^5$ Nmm (Fig. 16). The increased energy dissipation per cycle in the $c_{1M8-dhA}$ test is due to the different displacement histories: in displacement history A (d_{hA}) the initial displacement is larger ($d_0 = 10$ mm, Fig. 11) and therefore corresponds to a greater dissipation of hysteretic energy. Specimens exhibit a uniform trend when they are in the plastic regime (Fig. 16, from the 1st cycle for test $c_{1M8-dhA}$; from the 10th cycle for tests $c_{1M8-dhB}$ and $c_{2M8-dhB}$). The degradation of dissipated energy due to the pinching phenomenon is highlighted in Fig. 17, the dissipated energy in cycles subsequent to the first of the same amplitude, is not greater than 50% (Fig. 17), very low with respect to that of steel beams in three-point bending $\approx 100\%$ or diagonals of concentric X bracings $50\% \div 80\%$ [38], suggesting non-dissipative behavior.

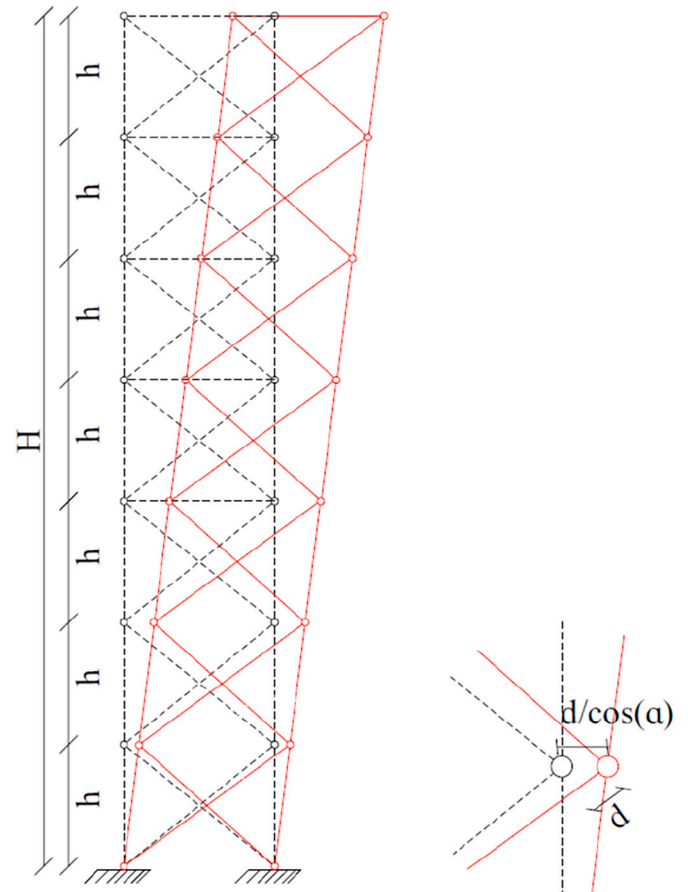


Fig. 10. Horizontal displacement at the brace-to-upright connection versus the horizontal displacement of the whole rack.

Table 4

Performed tests and loading protocol.

Test	Upright	Brace	Bolt	Test
$c_{1M8-dhA}$	110	$25 \times 25 \times 8$	M8	Cyclic - displacement history A
$c_{2M8-dhB}$	110	$25 \times 25 \times 8$	M8	Cyclic - displacement history B
$c_{3M8-dhB}$	110	$25 \times 25 \times 8$	M8	Cyclic - displacement history B

c_{Mj-dhz} : c = cyclic; i = number of the test; Mj = bolt diameter j; dhz = displacement history (z = A for displacement history A; z = B for displacement history B).

7. Development of the analytical model

7.1. Forces acting on the brace-to-upright joint

A theoretical analysis based on the Component Method (CM) [28,32] is conducted to estimate the initial elastic stiffness and strength of the frame connection with flanges connected. The CM can be applied to any kind of connection provided that the basic sources of strength and deformation are properly identified and modelled. The CM can be organized into three phases. The first is to identify the components in the connection, contributing to structural response. In the second phase each component is modelled by a force-displacement relationship. Bilinear elasto-plastic models (defined by an initial stiffness and an ultimate strength) are used for components that contribute to the stiffness

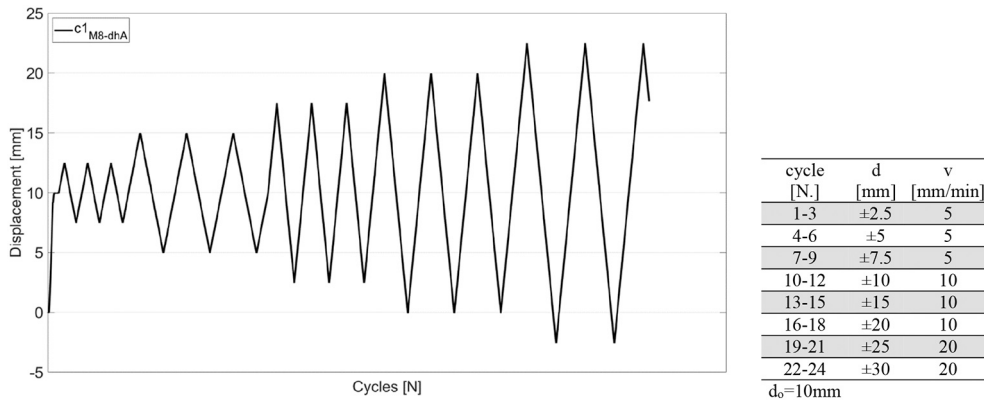


Fig. 11. Displacement history A – Test c1M8-dhA.

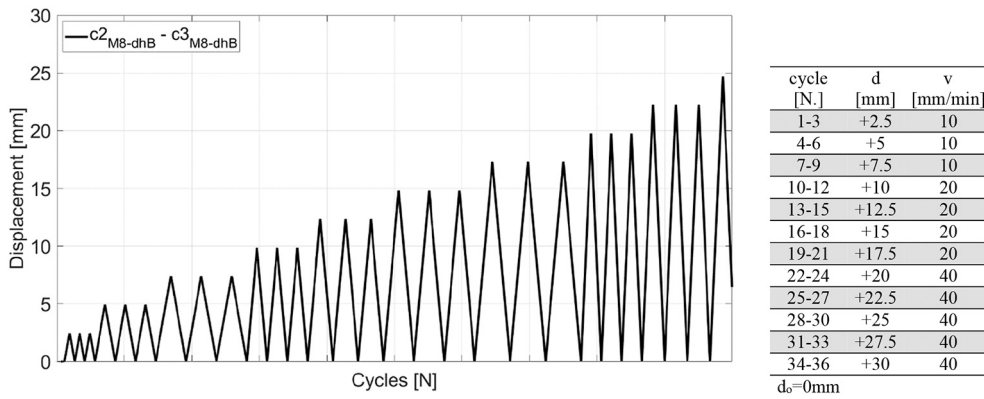


Fig. 12. Displacement history B – Test c2M8-dhB and Test c3M8-dhB.

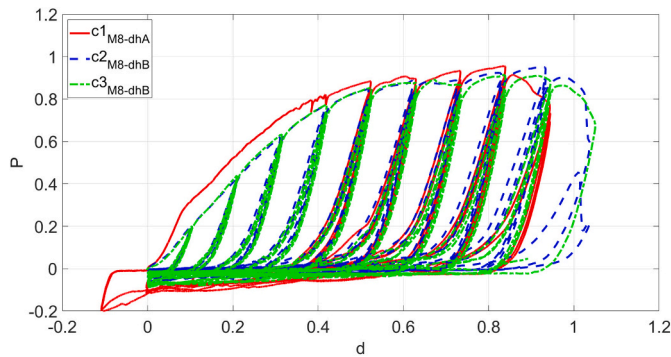


Fig. 13. Non-dimensional cyclic force-displacement curves.

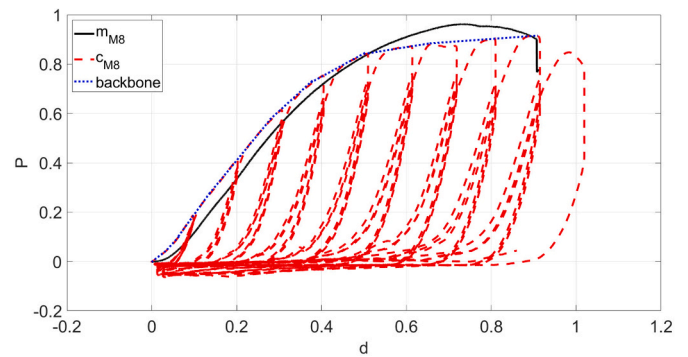


Fig. 14. Non-dimensional monotonic and cyclic force-displacement curves.

Table 5

Non-dimensional maximum load P_{exp} , and failure mode for all specimens subject to cyclic loading.

Test	P_{exp}	Failure mode
c1M8-dhA	0.95	Bearing of brace + bolt bending
c2M8-dhB	0.95	Bearing of brace + bolt bending
c3M8-dhB	0.91	Bearing of brace + bolt bending

and strength of the connection. In the last phase, these component models are introduced into the mechanical model of the overall connection with springs joined in series or parallel. The ultimate strength of the joint depends on the weakest component, and the initial elastic stiffness is influenced by each component [28].

The forces on the upright-to-brace connection are shown in Fig. 18. For equilibrium to be maintained, the horizontal component of the loads transferred by braces (see section A-A, Fig. 19) is equal to $T\cos(\alpha) = C\cos(\alpha)$. Thus, $T = C = N$. The load transferred by internal flanges of braces and acting on the bolt mid-span produces the bearing of the brace Fig. 19 a). The load transferred by brace external flanges and acting on the bolt-ends causes torsion in the upright and the bearing of the braces and the upright Fig. 19 b).

In the vertical direction, the vertical component of the load transferred by braces ($T\sin(\alpha)$ and $C\sin(\alpha)$) (section B-B in Fig. 18) balances the load on the upright ($Q = 2N\sin(\alpha)$).

The vertical component of the forces, transferred by braces, is applied at the bolt-ends Fig. 20 a) and at the bolt-mid span Fig. 20 b) and



Fig. 15. Test c2M8-dhB - Specimen deformation for different displacement steps.

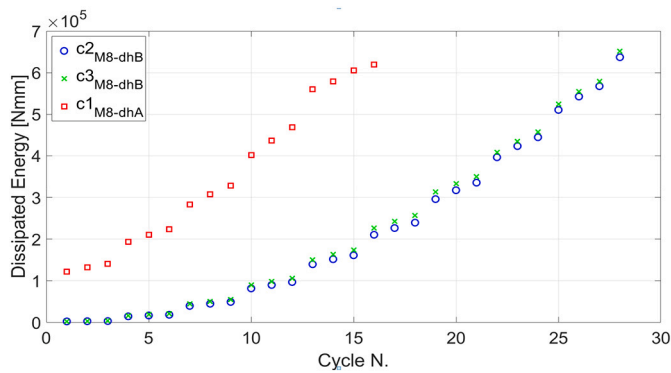


Fig. 16. Dissipated energy versus number of cycles.

results in the bearing of diagonals and the upright.

In addition, the component of the load transferred by the internal flanges of braces and acting on the bolt mid-span Fig. 20 b), produces the bending of the bolt, which is assumed to behave like a pinned-end beam loaded at mid-span.

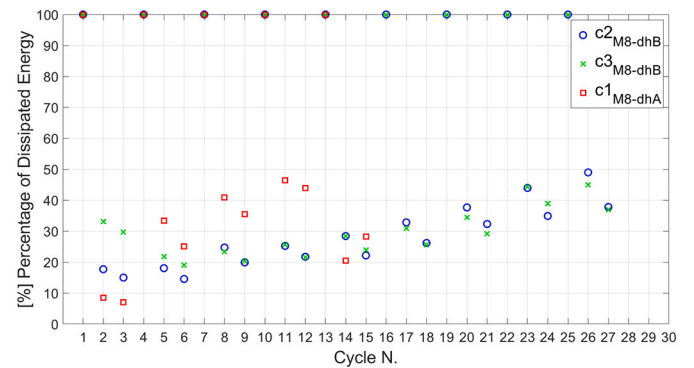


Fig. 17. Degradation of the energy dissipation capacity in the second and third cycle of each three-cycle series relative to the first cycle.

7.2. Evaluation of connection stiffness

In conventional frame analysis software used in engineering practice, the axial stiffness of the bracing members is a critical input parameter. This value is derived herein.

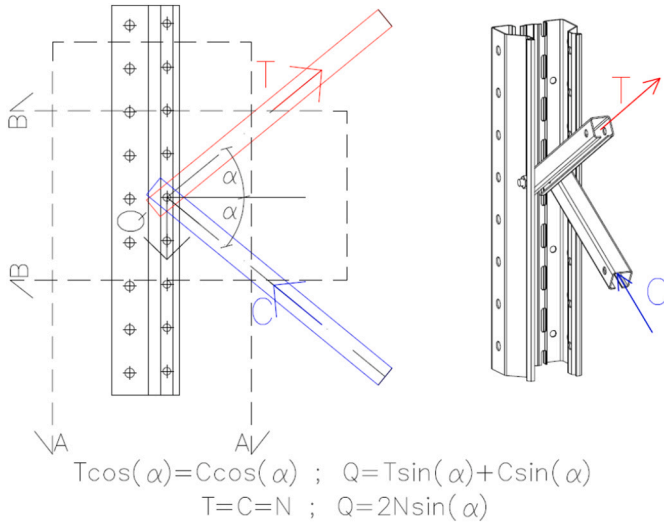


Fig. 18. Forces transferred by the upright frame bracing members to the joint.

Along the brace axis, the force $T = C = N$ (Fig. 18) produces the bearing strength of the diagonals. The stiffness due to the bearing of a single brace can be evaluated by Eq. (1) in accordance with Eurocode 3 [39]:

$$K' = K_{bearing,brace} = n_s 12k_b k_t d_b f_{u,brace} \quad (1)$$

where d_b is the bolt diameter, $n_s = 2$ is the number of shear planes (one on each flange of the single brace) and $f_{u,brace}$ is the ultimate strength of the upright.

Along the axis of the upright, the vertical component of the forces transferred by the braces produces the bearing of the upright and the bending of the bolt (Fig. 20).

The bearing stiffness of the upright can be found by Eq. (2)

$$K_{bearing,upright} = n_s 12k_b k_t d_b f_{u,upright} \quad (2)$$

where, d_b is the bolt diameter, $n_s = 2$ is the number of shear planes (one on each flange of the upright) and $f_{u,upright}$ is the ultimate strength of the upright.

The bolt can be modelled as a simply supported beam with the span equal to the distance between upright flanges L_b and loaded at mid-span with $Q/2 = N \sin(\alpha)$ (Fig. 21).

The translational stiffness of the bolt mid-section $K_{bending,bolt}$ can be calculated using Eq. (3):

$$K_{bending,bolt} = \frac{48EJ_b}{L_b^3} \quad (3)$$

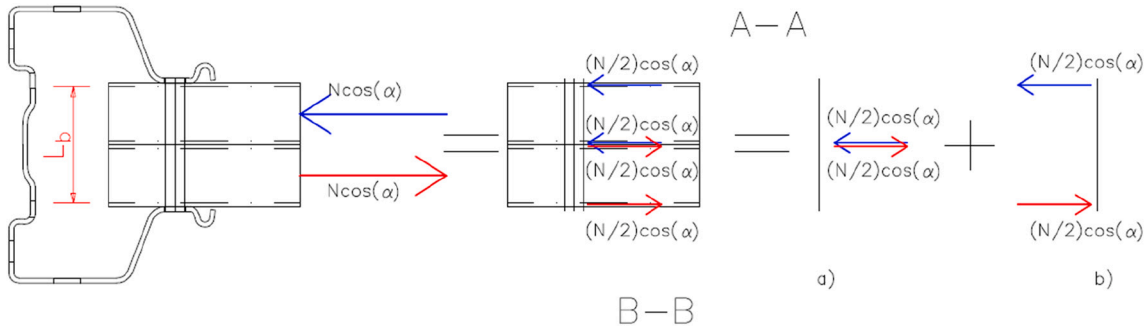


Fig. 19. Identification of the forces transferred by the upright frame bracing members on the joint members – Horizontal components.

a) forces transferred by internal flanges of braces and acting on the bolt mid-span producing the bearing of the braces.

b) forces transferred by brace external flanges and acting on the bolt-ends causing the torsion in the upright and the bearing of the braces and the upright.

where E is the Young modulus (210,000 N/mm²), J_b is the moment of inertia of the bolt section, and L_b is the distance between the upright flanges.

It should be noted that the resultant force along the vertical direction is $Q = 2N \sin(\alpha)$ (Fig. 20) which yields the bearing strength of the upright. The bending of the bolt is caused only by the vertical component of the load acting on the bolt mid-span, $Q/2 = N \sin(\alpha)$ (Fig. 21). Based on these considerations, the stiffness of the joint along the vertical direction K_v can be assumed (4) (Fig. 21):

$$K_v = \frac{K_{bearing,upright} 2K_{bending,bolt}}{(K_{bearing,upright} + 2K_{bending,bolt})} \quad (4)$$

To simplify the calculation, the contribution to the joint stiffness in the horizontal plane due to the effect of the upright torsion and the bearing of the column is neglected. In particular, the twisting moment (Fig. 19) is restrained by external supports and the upright torsional rigidity. Moreover, assuming a typical angle $\alpha \approx 45^\circ$, the component of the forces producing the upright bearing force along the horizontal direction ($N/2 \cos(\alpha)$ Fig. 19 b), is half of the vertical ($N \sin(\alpha)$, Fig. 20 b) and can be neglected.

Eq. (4) is evaluated assuming the axes of braces in the same plane of the bolt midspan. The stiffness K_v in Eq. (4) is defined along the line of action of the upright axis. The axial stiffness along the bracing member K'' is then determined. In Fig. 22, for one single bracing member, considering force and displacement components of the bracing member in the direction of the upright, the effective joint stiffness along the bracing member axis, inclined at angle α is given by Eq. (5):

$$K'' = \frac{K_v}{2 \sin(\alpha)^2} \quad (5)$$

where K'' is the joint stiffness in the direction of the bracing member.

Finally, for each single diagonal, the bearing stiffness of the brace K' (1) and the stiffness of the bolted connections along the brace axis K'' (5) due to the bearing of the upright and the bending of the bolt, are assembled. As depicted in Fig. 23, this leads to an equivalent stiffness K_{tot} acting along the member axis, which incorporates the connection flexibility as follows:

$$K_{tot} = \frac{K' K''}{(K' + K'')} \quad (6)$$

The stiffness of a single bolted connection Eq. (6) can be adopted to define a linear link formed by a linear elastic force-displacement curve with stiffness K_{tot} . The link can be easily introduced in an elastic finite element model with elastic frame elements used for beams and uprights as typically used to model the braced frame.

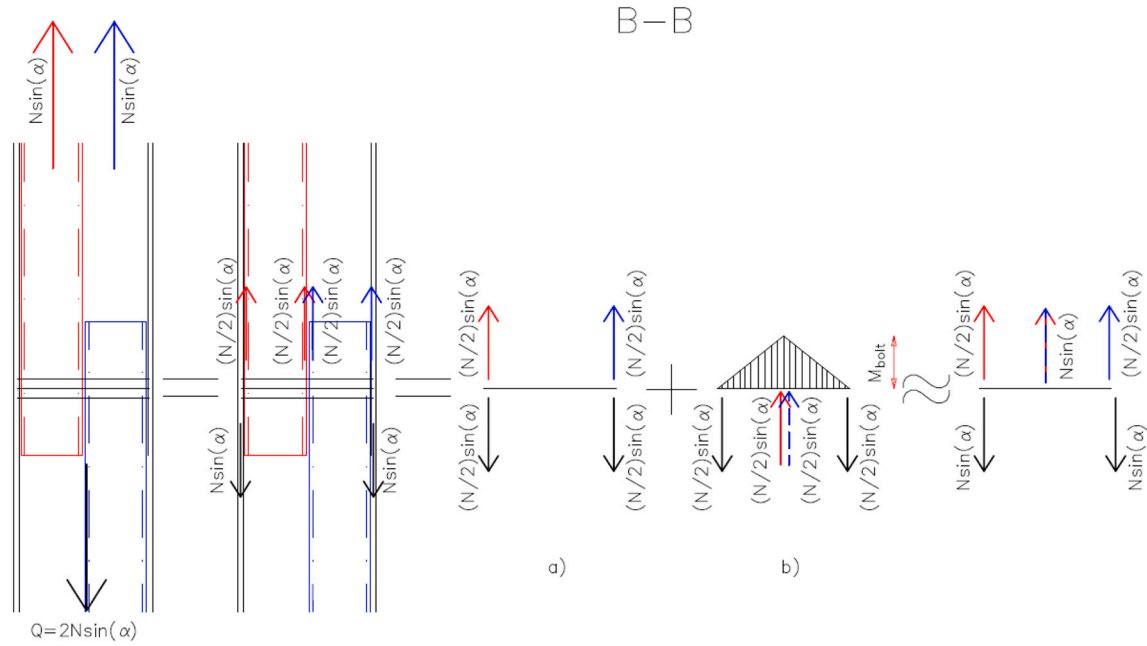


Fig. 20. Identification of the forces transferred by the upright frame bracing members on the joint members - Vertical components.

a) forces transferred by brace external flanges and acting on the bolt-ends causing the bearing of the braces and the upright.

b) forces transferred by internal flanges of braces and acting on the bolt mid-span producing the bearing of the braces and the upright, in addition to the bending of the bolt.

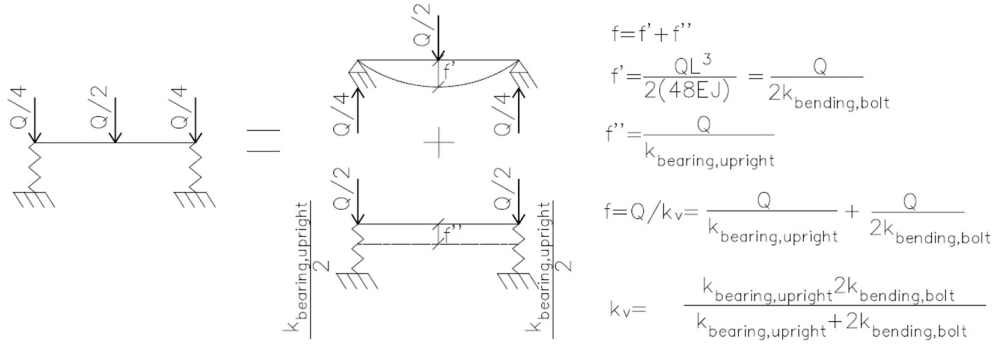


Fig. 21. Identification of the forces and deformations in the bolt.

7.3. Evaluation of connection resistance

The theoretical model is adopted to predict the resistance and failure mode of the connection. The connection resistance is taken as the resistance of the weakest component.

The shear resistance of the bolt can be found by Eq. (7) [39]:

$$F_{v,th} = \alpha_v f_{u,b} A_b \quad (7)$$

where $\alpha_v = 0.6$ for bolts belonging to class 8.8 [39], A is the tensile stress area of the bolt and $f_{u,b}$ is the ultimate tensile strength of the bolt.

The bearing resistance of each member can be evaluated according to Eq. (8) [39]:

$$F_{b,th} = k_t k_s d_b t_b f_b \quad (8)$$

where t_b and f_b are the thickness and ultimate strength of the connected member, respectively and d_b is the bolt diameter.

The design force producing the ultimate moment on the bolt,

assumed as a pinned-ends beam, can be evaluated by Eq. (9):

$$F_{M,th,bolt} = \frac{4M_{th,bolt}}{L_b \sin(\alpha)} \quad (9)$$

where $M_{th,bolt}$ is the ultimate moment of the bolt cross-section.

8. Comparison

The effectiveness of the proposed approach based on the Component Method is evaluated by comparing the results of the simulated experimental (exp) tests with the numerical results obtained through two numerical models (rig) and (ana). FE numerical models are developed in SAP2000 [40], a program, adopted in common practice, which offers 6 degrees of freedom (DOFs) beam formulations, neglecting the cross-section warping which affects mono-symmetric cross-sections, such as the uprights of the case study [41]. In FE models, the upright and the braces are modelled by linear elastic “Frame” elements [40]. The elastic

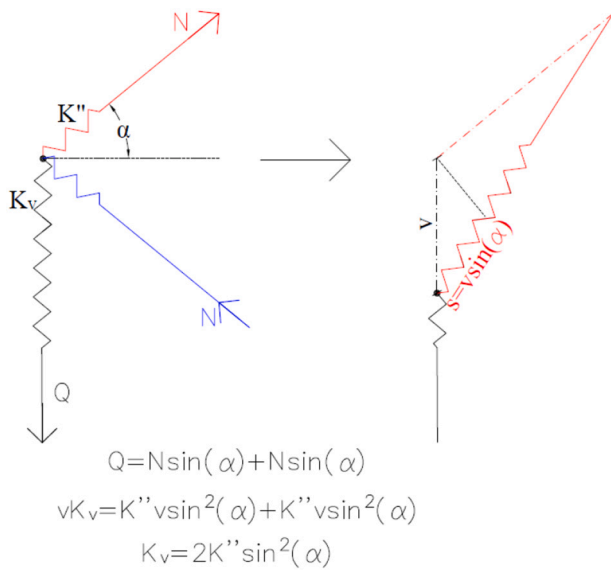


Fig. 22. Derivation of connection stiffness along the bracing member.

modulus and Poisson's ratio are taken as 210,000 N/mm² and 0.3, respectively. Boundary conditions are simulated by applying appropriate constraints: a hinge at one upright end and at the short brace end and releasing only the vertical displacement at the long brace end (Fig. 24). In the first model, the connection is assumed rigid (*rig*) as commonly done in common practice. In the second (*ana*), the actual behavior of the brace-to-upright joint, evaluated by the analytical method, is simulated introducing a "2 Joint Link" element [40], at the ends of each brace, that simulates the stiffness of the connection and its effective resistance. The length of the "2 Joint Link" element, introduced in the analytical model (*ana*) has been chosen as the distance from the lips of the upright to the bolt, which joins braces to the upright. In the model of the overall bracing frame, the eccentricity between the upright axis and the connection can be modelled by an infinitely rigid resistance element. The "2 Joint Link" element is a spring with a force-displacement relationship defined by a linear elastic curve identified by the linear elastic stiffness evaluated by Eq. (6) and the ultimate strength given by the minimum of Eqs. (7)–(9). Non-linear static analyses, considering large displacements and second-order effects are

performed, increasing the vertical displacement at the control point, at the long member-end, until the system fails.

As can be seen from Table 6, results indicate a significant improvement in predicting the stiffness and ultimate load; the ratio of numerical to experimental stiffness is reduced from 13.71 (K_{rig}/K_{exp}), obtained in elastic models where the local response at the bolted joint is not considered, to 1.13 (K_{ana}/K_{exp}). The proposed model, using only "Frame" elements with a linear-elastic material equipped with the proposed link, is thus able to accurately predict the rack connection stiffness. The slight difference in the value of the numerical stiffness can be caused by the twisting of the upright, which is neglected in the analytical model.

The analytical numerical model can predict the failure mode of the connection within $\pm 11\%$ as shown in Table 7. As observed in the experimental tests, in the bracing arrangement with an M8 bolt, the failure mode is a flexural plastic hinge developed in the middle of the bolts. Collapse is due to the bearing of braces when an M10 bolt is used. The same results are obtained from the analytical numerical model (Table 7), in which the weakest component, which fails and determines the ultimate resistance of the joint is highlighted in grey ($F_{M,th,bolt}$, which corresponds to the force producing the ultimate moment of the bolt, for bracing arrangement with an M8, and $F_{b,th,braces}$, which corresponds to the bearing resistance of the brace, for bracing arrangement with an M10).

The model will enable innovation and improved connection response as geometry and material properties can be easily varied to produce an optimum solution.

Comparison of different analysis results with experimental values for the stiffness of braced frames used in industrial racking shows that analysis results can be grossly in error if the joint flexibility of bracing members is ignored, approximately $13\times$ the experimental connection stiffness. The resistance of the joint depends on the bolt diameter adopted to connect the upright with the braces and can be only described by introducing the proposed link at brace-ends. In the absence of experimental data, the analytical model can produce reasonable simulations, and because the modelling approach is high-throughput, can enable parametric variation with little computational expense.

9. Conclusions

This study investigates the structural response of the joints of braced cross-aisle frames of thin-walled cold-formed steel storage racks to determine their stiffness and resistance. Braces which are formed by connecting two members flange-to-flange demonstrate good

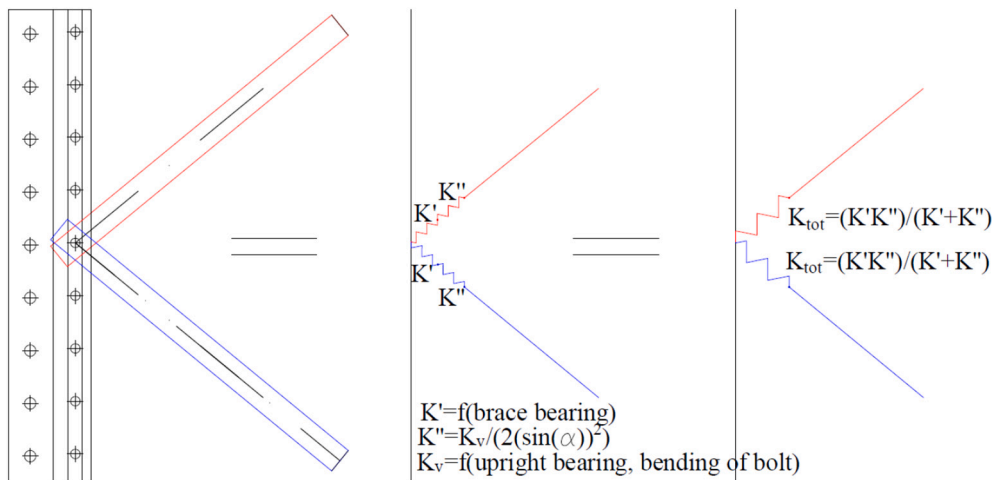


Fig. 23. Upright-to-braces connection stiffness.

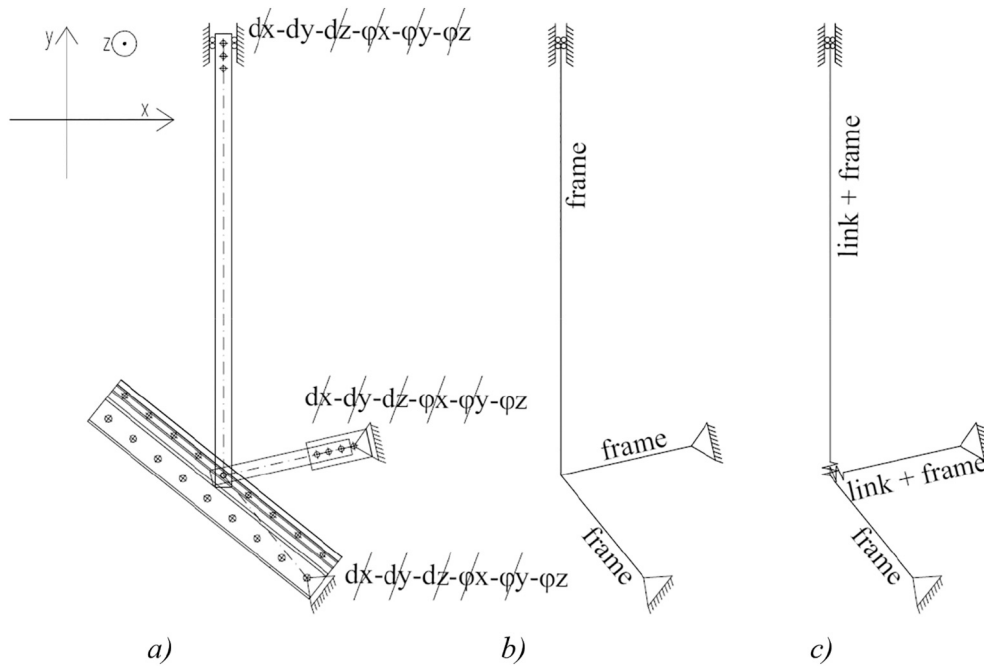


Fig. 24. Upright-to-braces connection. a) Experimental test, b) common FE numerical model ($_{rig}$), c) FE numerical model ($_{ana}$) with links to simulate the connection structural response.

Table 6

The analytical stiffness considering the joint flexibility K_{ana} and the stiffness obtained through a rigid model K_{rig} , divided by the experimental elastic stiffness K_{exp}

Test	K_{rig}/K_{exp}	K_{ana}/K_{exp}
m1 _{M8}	14.21	1.13
m2 _{M8}	14.21	1.16
m3 _{M8}	14.21	1.14
m1 _{M10}	12.21	1.07
Mean	13.71	1.13

performance, and were adopted herein. Monotonic and cyclic experimental testing was used to assess the structural response of the joints while capturing local effects. The monotonic and cyclic backbones agree well, though the cyclic response is highly pinched. Energy dissipation analysis of the per-cycle dissipation suggests that the behavior of the joint is non-dissipative. The experimental results were used to compare the accuracy of different methods of analysis for establishing the structural performance of rack frames. The analyses considered include

two elastic FE models consisting of elastic frame elements. The first, frequently used by practitioners, has rigid joints. In the second, the local response of the connections is modelled by a force-displacement link whose properties were determined via the Component Method. The comparison of results showed significant discrepancies that were attributed primarily to the bracing member connection flexibility. An elastic rigid model overestimates the flexibility by nearly 14 times and is not able to capture the failure of the connection component. Introducing the linear link results in a stiffness 1.13 times greater than the experimental stiffness, representing significant improvement. Moreover, the analytical model can predict the failure mode and the resistance of bracing members within $\pm 11\%$. This analytical evaluation of the ultimate load and connection stiffness could be an attractive approach for practicing engineers. By highlighting the weakest component of the connection and the influence of each design parameter, the method can be adopted to increase the structural performance of bracing-upright connections.

Table 7

The theoretical resistance of each component divided by the maximum experimental load P_{exp} , with the identification in grey of the weakest component and the connection failure mode.

TEST	$F_{v,th,bolt}/P_{exp}$	$F_{b,th,brace}/P_{exp}$	$F_{M,th,bolt}/P_{exp}$	Failure mode
m1 _{M8}	1.52	1.04	0.86	Bolt collapse
m2 _{M8}	1.70	1.16	0.96	Bolt collapse
m3 _{M8}	1.45	0.99	0.82	Bolt collapse
m1 _{M10}	1.99	1.06	1.22	Bearing of brace
c1 _{M8-dhA}	1.59	1.08	0.90	Bolt collapse
c2 _{M8-dhB}	1.60	1.09	0.90	Bolt collapse
c3 _{M8-dhB}	1.67	1.13	0.94	Bolt collapse
Mean		1.06	0.89	

Credits

The experimental campaign was performed in collaboration with the company ROSS SpA, Scarperia and San Piero, Florence (IT), under the supervision of M. Orlando.

Declaration of Competing Interest

No.

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