Factors contributing to the transverse shear stiffness of bolted cold-formed steel storage rack upright frames with channel bracing members

Nima Talebian⁽¹⁾, Benoit P. Gilbert⁽¹⁾, Nadia Baldassino⁽²⁾ and Hassan Karampour⁽¹⁾

⁽¹⁾ School of Engineering and Built Environment, Griffith University, Gold Coast, Australia

⁽²⁾ Department of Civil, Environmental and Mechanical Engineering, University of Trento, Trento, Italy

ABSTRACT:

Accurately determining the transverse shear stiffness of steel storage rack upright frames is essential in calculating the elastic buckling load, performing earthquake design and serviceability checks. International racking design specifications recommend different approaches to evaluate this stiffness. The Rack Manufacturers Institute (RMI) specification conservatively uses an analytical solution based on Timoshenko and Gere's theory while the European (EN15512) and Australian (AS4084) specifications recommend experimental testing to be conducted. Previous studies have shown that Finite Element Analyses (FEA), solely using beam elements, fail to reproduce experimental test results and may overestimate the transverse shear stiffness by a factor up to 25. This discrepancy is likely attributed to the local deformations occurring at the bolted joints. In this paper, a model to capture the transverse shear stiffness of upright frames is developed using shell elements and advanced FEA. Its accuracy is verified against published experimental test results performed on three commercially used upright frame configurations with lip-to-lip bracing pattern. The model accurately reproduces the experimental stiffness, with differences ranging from 2% to 17%. Based on the FE model, the factors contributing to the transverse shear deformation of the frames are quantified and discussed for both lip-to-lip and back-to-back bracing patterns. For lip-to-lip upright frames, results show that the local deformations at the end of bracing members contributes the most to the shear deformation of the frames. For back-to-back upright frames, bolt bending and axial deformation of braces contribute the most to the shear deformation of the frames. Results from this paper would assist the racking industry in improving their products by focusing on the factors influencing the most the behavior of the frames.

27 *Keywords:* Steel storage racks, Cold-formed steel, Upright frames, Shear stiffness, Finite Element
28 Analysis

29 **1. Introduction**

30 Steel storage rack systems, commonly assembled from cold-formed steel profiles, are extensively
31 used in industry to store goods. Goods are placed on pallets which are positioned on the racks using
32 forklift trucks. They act as freestanding structures and are designed as lightly as possible, while still
33 capable of carrying heavy loads [1]. Their popularity lies in their ability to increase storage capacity
34 by both minimising the floor space and providing a number of different storage configurations [2].
35 The most common type of rack is referred to as “selective” storage rack and typically consists of
36 uprights, pallet beams, bracing members and connectors, as illustrated in Fig. 1 In the down-aisle
37 direction, the stability of unbraced racks is solely ensured by the base plate-to-floor and pallet beam-
38 to-upright semi-rigid connections [3-6]. In the cross-aisle direction, the stability is ensured by the
39 upright frames, each consisting of two uprights connected by bracing members, as shown in Fig. 1
40 and Fig. 2. The bracing members are commonly cold-formed lipped channel-sections bolted to the
41 upright flanges. Welded connections are also encountered. Other forms of cold formed profiles, such
42 as circular hollow sections, are also used in practice.

43 Although, the configuration of steel storage rack structures is simple, as they are assembled from
44 beams, uprights and bracings, their analysis and design are complicated. Due to the nature of the cold-
45 formed steel elements, their performance is influenced, among others, by local deformations at the
46 uprights and bracing members at the connections [7]. The base plate-to-floor and beam-to-upright
47 semi-rigid connections also influence the structural behaviour of the system [8, 9].

48 The transverse shear stiffness of the upright frames has a significant impact on the behaviour of the
49 overall structure in the cross-aisle direction. As rack structures are sensitive to second order effects,
50 precisely determining the shear stiffness is of a great importance for serviceability checks, and to
51 calculate the elastic buckling loads and earthquake design forces. This is especially relevant for high-
52 bay racks that can reach heights greater than 20 metres and racks supporting the building enclosure
53 which are subjected to cross-aisle horizontal forces due to wind loading. The shear behaviour of the

upright frames is currently poorly understood and investigations are still needed to advance the knowledge.

To understand all factors affecting the transverse shear behaviour of bolted cold-formed steel storage rack upright frames, numerical models able to capture the deformations at the bolted connections are required [1, 7, 10]. This paper develops and details an advanced Finite Element (FE) model to accurately predict the transverse shear stiffness of storage rack upright frames with channel bracing members. The accuracy of the model is verified against experimental tests performed on three frame configurations in [1] and tested following the two different set-ups. The various factors influencing the shear stiffness of the rack upright frames with lip-to-lip and back-to-back channel bracing members are then numerically identified, quantified and discussed. These results would provide rack manufacturers the possibility to improve their design by targeting the factors influencing the most the shear stiffness of the frames.

2. Literature review

2.1 Design Specifications

Currently, different approaches are adopted by the three main international storage rack design specifications AS4084 [11], EN15512 [12] and RMI [13] to determine the shear stiffness of cold-formed steel storage rack upright frames. Timoshenko and Gere's [14] theory is used in the Rack Manufacturers Institute specification [13] in which the upright frame shear deformation is assumed to purely arise from the axial deformation of the bracing members. In the European design specification [12], experimental testing of the frame in the longitudinal direction is proposed to determine the stiffness per unit length. The test set-up consists of an upright frame restrained from rigid body rotation and with at least two bracing panels. One of the uprights is pinned at one end and a longitudinal force F is applied at the opposite end of the other upright through its centroidal axis, as depicted in Fig. 3. The longitudinal displacement of the upright, on which the force is applied, is recorded and the slope k_{ti} of the linear part of the experimental load-displacement curve is calculated.

79 The stiffness k_{ti} is then used in conjunction with Timoshenko and Gere's [14] theory to estimate the
80 upright frame transverse shear stiffness S_{ti} as,

$$81 \quad S_{ti} = \frac{k_{ti} d^2}{h} \quad (1)$$

82 where d is the distance between the centroidal axes of the two uprights and h is the height of the
83 frame.

84 The Australian Standard AS4084 [11] proposes the use of the testing method suggested by the
85 European design specification EN15512 [12] and an alternative approach in which the frame is loaded
86 in the transverse direction to evaluate the combined shear and bending stiffness, as shown in Fig. 4.
87 In the alternative approach, the frame is composed of a minimum number of two bracing panels. The
88 bottom ends of the uprights are pinned to a rigid frame and the top ends of the uprights are prevented
89 from out-of-plane displacements. A load F is applied to the frame at the elevation of the top horizontal
90 bracing member. Two displacement transducers are positioned at the elevations of the top (i.e. where
91 the load F is applied) and bottom horizontal bracing members. The combined bending and shear
92 transverse stiffness S_{cti} is then calculated as,

$$93 \quad S_{cti} = k_{cti} h \quad (2)$$

94 where k_{cti} is the slope of the experimental load-displacement curve, with the displacement being
95 calculated as the difference between the two transducers, and h is height of the frame.

96 *2.2 Frame behaviour*

97 Very limited studies have investigated the transverse shear stiffness of cold-formed steel storage rack
98 bolted upright frames. Rao et al. [15], Sajja et al. [7] and Sajja [10] experimentally and numerically
99 investigated the shear stiffness of various upright frames. Developed models with beam elements
100 which included upright bending stiffness, the eccentricity of bracing members and the bending of the
101 bolt connecting the bracings to the upright, were not able to successfully reproduce the experimental

102 test results. The models overestimated the shear stiffness by a factor of 2 to 5. The discrepancy was
103 attributed to the “torsional distortion” of the uprights, not being considered in the model. It was
104 recommended that it is essential to consider the contact behaviour between various elements at the
105 connections to accurately determine the shear stiffness.

106 To accurately reproduce the frame behaviour, the influence of warping effects must be considered in
107 FEM modelling of mono-symmetric rack upright sections. This can be done using 7DOFs beam
108 elements [16, 17, 18, 19] or 3D modelling the uprights with either shell or solid elements.

109 Gilbert et al. [1] performed 36 tests of bolted upright frames following both the EN15512 [12] and
110 alternative AS4084 [11] test set-ups. The two methods were compared and the practical use of the
111 alternative method proposed by the AS4084 was demonstrated. The conclusions were based on three
112 upright section types and two different bracing cross-sections (CHS and channels), totalling six
113 different upright frame types. Finite Element models were also developed in Gilbert et al. [1] using
114 beam elements. Warping effects were considered for the uprights using 7DOFs beam elements. The
115 analyses overestimated the upright frame shear stiffness by a factor of 9 to 25 for the test set-up in
116 the EN15512 [12] and 3 to 16 for the alternative test set-up in the AS4084 [11]. This difference was
117 mainly attributed to the local deformation of the connections between bracing members and uprights,
118 not being considered in the model.

119 Sangle et al. [20] conducted elastic stability analysis of upright frames through 3D finite element
120 analysis and emphasized the need for 3D modelling to study the behaviour of racking systems. Far et
121 al. [21] numerically and experimentally investigated the shear behaviour of the upright frames.
122 Developed FE models of the upright frames using solid elements overestimated the shear stiffness by
123 30%. A simplified modelling approach was also proposed to account for the flexibility in bolted
124 connections. Roure et al. [22] experimentally and analytically studied the behaviour of upright
125 frames. Tests were performed on the joints between uprights and braces as well as on the upright
126 frames. A simple practical design approach was proposed based on an adjusted cross-sectional area
127 at both ends of the bracing members to consider that the axial stiffness of the bracing members is

128 affected by the local distortions at the joints. Finite element modelling of storage rack frames using
129 shell elements was performed and validated against test results by Cardoso and Rasmussen [23]. The
130 semi-rigid behaviour of connections was implemented in the model by moment-rotation curves
131 obtained from available component tests.

132 **3. Experimental tests used to verify FE models**

133 As mentioned in the literature review, Gilbert et al. [1] reported tests performed on three different
134 upright frames with channel bracing members to determine both the shear stiffness S_{ti} (Eq (1)) and
135 the combined transverse bending and shear stiffness, S_{cti} (Eq (2)). In total 17 tests were performed
136 both in Australia and Italy with channel bracing members. These tests are used in the present paper
137 to validate the FE model. The upright frames were assembled from three upright types (referred to as
138 “Type A”, “Type B” and “Type C”) and with lip-to-lip channel-sections in a Z-pattern, as shown in
139 Fig. 5. Height of the tested upright frames was equal to 3,432 mm. Depending on the frame
140 configuration, “d” was equal to 948.9, 953.1 and 933.5 mm for Type A, Type B and Type C upright
141 frames, respectively. Each configuration was tested following both the EN15512 [12] and the
142 alternative AS4084 [11] test set-up procedures. Three repeat tests have been performed for each
143 upright frame configuration. When tested Type A upright frame following the EN15512 [12], only
144 two tests were performed. The main dimensions of the uprights used are shown in Fig. 6 and the
145 cross-sectional properties of the uprights and bracing members are presented in Table 1. Type A
146 upright frames were tested with bracing type C35x20x1.2, while Types B and C upright frames were
147 tested with bracing type C35x35x1.5. End plates were welded to the ends of the uprights to ensure
148 easy connection and restrain warping at these locations. For the frames tested following AS4084 [11]
149 displacement was recorded at each bracing elevation, from LVDT 4 at the bottom bracing elevation
150 to LVDT 1 at the top bracing elevation, as shown in Fig. 5.

151 The tests were performed with two different boundary conditions, depending in which country the
152 tests took place. The test set-up for all the upright frames tested following the alternative method in
153 the AS4084 [11] and Type A upright frame tested following the method in the EN15512 [12] is shown

154 in Fig. 7 (a). In these tests, the frames were restrained from lateral out-of-plane displacements at six
155 locations along the height of the frame, as depicted in Fig. 7 (a). Fig. 7 (b) and (c) shows the vertical
156 restraint and their schematic view, respectively. Bottom Nylon pads sprayed with silicone were placed
157 underneath the uprights so as to allow the horizontal displacement of the frame. The top Nylon pads
158 were pinned above the centroid axis of the uprights. Specifically, the latter pads were loosely
159 connected to steel square hollow sections (SHS) using steel balls to both avoid out-of-plane
160 movement and allow the uprights to rotate. The supported steel SHS in Fig. 7 were bolted to the
161 strong floor through threaded bars.

162 On the other hand, Fig. 8 (a) shows the experimental test set-up for Types B and C upright frames
163 following the method in the EN15512 [12] in which the frames were restrained at five locations to
164 prevent out-of-plane displacement. Fig. 8 (b) shows a schematic view of the vertical restraint at four
165 locations along the loaded upright. Steel plates were placed underneath the upright and a non-
166 frictional material was inserted between the steel plate and the upright so as to allow the longitudinal
167 displacement of the upright. The gap between the upright and top steel plate was to allow the upright
168 to twist without excessive out-of-plane displacement. Fig. 8 (c) shows a schematic view of the vertical
169 restraint at free end of the pinned upright. This restraint prevented out-of-plane displacement,
170 horizontal displacement, and rotation about the upright longitudinal axis.

171 To ensure, full contact between all elements in the experimental tests, preloading was applied. For
172 Type B and C upright frames tested following the EN15512 setup, the load was applied up to 6 kN,
173 reversed to 0.5 kN and increased up to failure. For the tests performed following the AS4084 test
174 setup and Type A upright frames following the EN15512 test setup, the applied load was increased
175 to either 1 kN or 2 kN (depending on the frame configuration – see [1]) and 3 kN, respectively, and
176 then reduced to 0 kN before loading to failure.

177 **4. Finite element modelling**

178 A FE model built using the Finite Element software ABAQUS [24] is detailed hereafter to capture
179 both the global and local deformations of the upright frames.

180 4.1. Geometry, element types and mesh

181 Reduced integration four node shell elements, S4R, were used to model the uprights, upright end
182 plates and bracing members at their wall centre-line. The use of shell elements will allow warping
183 effects to be considered in the model. Only largest perforations of the uprights were considered in the
184 FE model. Bolts, Nylon pads, steel plates and pinned supports were modelled using C3D10 solid
185 elements. Based on convergence studies performed on a single upright and bracing member, the size
186 of the S4R elements was found to be about 10 mm $\times$ 15 mm for the uprights and 10 mm $\times$ 10 mm for
187 the bracing members to well capture the frame deformation. In the vicinity of the bolted connections,
188 3 mm $\times$ 3 mm mesh size was found to be fine enough to accurately capture the local deformations of
189 the connections. The mesh size was further refined locally around the bolt holes to account for the
190 presence of stress concentrations [25]. Five integration points through the thickness of the shell
191 elements were considered.

192 4.2. Boundary conditions

193 Due to the loose connection between the top Nylon pads and the uprights (Fig. 7), the influence of
194 the top pads on FE response can be physically ignored for all the upright frames tested following the
195 alternative method in the AS4084 [11] and Type A upright frame tested following the method in the
196 EN15512 [12]. Only bottom Nylon pads were consequently modelled in the FEA simulations and
197 fixed to the ground. The interaction between the uprights and bottom pads was modelled using a
198 frictionless surface-to-surface contact. During tests, upright frames rotate and displace towards the
199 bottom Nylon pads and remain in contact with them. Therefore, only modelling the bottom pads is
200 enough to prevent out-of-plane displacements and this was found to accurately represent the effect of
201 vertical restraint on the frame response, as detailed in Section 5. To apply the lateral load, a rigid plate
202 was tied to the upright frames at the point of the application of the load to prevent local deformation.
203 Fig. 9 shows the details of the vertical restraints used in the FE models.

204 As shown earlier in Fig. 8, the vertical displacement of the frames was restrained at five locations for
205 Types B and C upright frames tested following the methodology in the EN15512 [12]. Fig. 10 (a) and

(b) details the vertical restraint used in the FE model. For the four restraints located on the loaded upright, steel plates fixed to the ground. A frictionless surface-to-surface contact was used to model the interaction between the upright and the steel plates, as shown in Fig. 10 (a). To prevent twisting, transvers and vertical displacement, the free end of the pinned upright was restrained at specified points, as shown in Fig. 10 (b).

4.3. Upright and bracing member bolted connections

To replicate the actual behaviour at the bolted connections, the interaction between elements was modelled using contacts as shown in Fig. 11. Especially, the contacts between (i) the bolts and the bolt holes, and (ii) upright flange edges and bracing webs were modelled using the node-to-surface discretization method, with small sliding and zero initial clearance. As noted earlier, the looseness in the connections can be ignored due to the preloading applied before commencing each test so as to ensure that all elements were in contact at the beginning of the test. The contacts between (i) the bracing members and upright flanges, and (ii) the bolt head/nut and uprights were simulated using the surface-to-surface discretization method with finite sliding. Hard contact with friction coefficient equal to 0.3 was considered as interaction behaviour for all contacts [26]. The bracing members were assumed to fit perfectly between the upright flanges, i.e. no gap was considered between these elements.

4.4. Material properties

The material non-linearity was modelled using the von Mises yield criteria and isotropic hardening. The stress-strain curves were inputted into ABAQUS as multi-linear curves and derived from tensile tests on coupons cut from the uprights and bracing members. To account for the change of cross-sectional dimensions of the coupons during testing, true engineering stress and strain were employed in the numerical model. The nominal stress (σ_n) and strain (ε_n) are converted to true stress (σ_t) and strain (ε_t) using the following equations [27, 28]:

$$\sigma_t = \sigma_n(1 + \varepsilon_t) \quad (3)$$

$$\varepsilon_t = \ln(1 + \varepsilon_n) \quad (4)$$

Fig. 12 shows the $\sigma_i - \varepsilon_i$ curves used for all uprights and bracing members. Bolts were modelled using an elastic perfectly plastic material with yield stress equal to 640 MPa [29]. All other components including upright end plates and pinned supports (test rig) were modelled as elastic materials. Table 2 summarises the average measured material properties for all upright types and bracing members. Three tensile tests were performed per upright type and bracing member. The Poisson's ratio was assumed equal to 0.3.

5. Validation of the FE model and comparison with experimental results

To consider all deformations occurring in the frames, nonlinear geometry and material analyses were ran. Yielding at the bolt holes were found to occur at an early stage of loading (particularly for the frames tested following the alternative AS4084 test set-up) and this is captured by the FEA. As the paper focuses on the initial shear stiffness of the frames, analyses were stopped when the applied load reached about 65% (AS4084 test set-up) and 60% (EN15512 test set-up) of the experimental failure load. Note that in terms of failure, the models predict that, similar to the experimental results, Type A upright frame fails in flexural-torsional buckling of the bracing members while Type B and C upright frames fail by shearing of the bracing members at the bolt hole. However, due to large local deformations at the bolt holes for Type B and C upright frames, the FEA stopped converging before reaching the peak load. Further investigations would be needed outside the scope of this paper to improve the convergence of the models and consider the high level of nonlinearity involved.

5.1. Tests following the AS4084 test set-up

Fig. 13 to 15 compare the load-deflection curves (deformation taken as the difference between the displacements at the top and bottom bracing elevations - Fig. 4) between FEA and experimental test results for the three upright frames configurations. The initial nonlinearity in the experimental load-deflection curves in Figs. 13 to 15 is related to the preloading and unloading phases applied to the frames before starting the tests as detailed in Section 3. The initial nonlinear response observed in the FEA load-deflection curve in Fig. 13 is attributed to nonlinear frictional contacts in the model. Table 3 gives the stiffness k_{cti} (Eq. (2)) calculated from both the FEA and experimental results. The stiffness

is calculated by performing a linear regression on the load-deflection curves, i.e. between 1 kN and 3 kN for Type A and C upright frames, and between 1 kN and 2.5 kN for frames with Type B upright frames. Fig. 13 to 15 and Table 3 show a reasonable agreement between the FEA and experimental tests. The average difference in stiffness is 7% (with Coefficient of Variation (CoV) equal to 11%) and reaches up to 17% for Type B upright frame. In terms of accuracy, the present model represents a significant improvement when compared to published models [1, 7, 21] mentioned in Section 2.2. Fig. 16 compares the displacements recorded by all LVDTs (Fig. 5) (i.e. at all bracing elevations) with FEA results for the 1st test performed on Type A upright frame. The FEA is able to well predict the overall frame displacement. Fig. 17 compares the FE deformation of the frame to available experimental photos of tested Type A upright frame. The developed FE model predicts well the overall deformed shape. This upright frame fails in flexural-torsional buckling of the diagonal bracing member in compression. Similarly, Fig. 18 compares the FE and experimental overall deformation of Type B upright frame. Due to the eccentricity between the uprights shear centre and the bracing members at the joints in lip-to-lip configuration, the uprights twist about their longitudinal axis. Fig. 19 (a) and (b) compare the FE and experimental overall deformation of Type C upright frame and Fig. 19 (c) and (d) compare the FE and experimental local deformations at the bolted connections for the latter upright. Similar to Fig. 17, Fig. 18 and 19 show that FEA is able to well capture the global and local behaviour of the upright frames. The rotations of the top of the frames is also well captured by the FE model, as shown in Figs. 18 and 19. Note, to magnify the deformation of the frame, Fig. 17 to 19 were shown at a FE displacement greater than the one shown in Fig. 13 to 15.

5.2. Tests following the EN15512 test set-up

Fig. 20 to 22 compare the load-deflection curves, at the load application point of the EN15512 test set-up, between the FEA and experimental test results for the three types upright frames configurations. The initial nonlinear responses observed in the FEA load-deflection curves are attributed to nonlinear frictional contacts in the model, but it manifests differently to Fig. 13 due to

the different boundary conditions. Table 3 gives the stiffness k_{ii} (Eq. (1)) calculated for both the FEA and experimental results. In Table 3, the stiffness were determined by performing a linear regression on the load-deflection curves, specifically between 5 kN and 12 kN for Type A upright frame, between 2 kN and 8 kN for Type B upright frame, and between 4 kN and 14 kN for Type C upright frame. Fig. 20 to 22 and Table 3 show a good agreement between the FEA and experimental tests. The average predicted to experiment ratio is 1.00 (with CoV of 6%) and the maximum ratio is 1.07 for Type A upright frame.

Fig. 23 presents the deformed shape of the FEA and experimentally tested Type A upright frame and shows that the developed FE model predicts well the overall deformed shape. Similar to the AS4084 test setup, this upright frame fails in flexural-torsional buckling of the diagonal bracing members in compression. Fig. 24 shows the upright rotation (due to eccentricity between the uprights shear centre and the bracing members at the joints in lip-to-lip configuration) and the deformation of the bracing members at the bolted connections for Type B upright frame. Similarly, Fig. 25 shows the local deformation at the bolted upright-bracing connection for Type C upright frame. Figs 24 to 25 also show that FE models are able to well capture the global and local deformation of the upright frames when tested following the EN15512 test set-up. Note, to magnify the deformation of the frame, Fig. 23 to 25 were shown at a FE displacement greater than the one shown in Fig. 20 to 22.

6. Contribution of factors affecting the transverse shear stiffness

6.1 General

The verified FE model is used herein to determine the contribution of factors affecting the shear stiffness of the frames. The following factors are investigated (a) bolt bending deformation, (b) local deformation at the bracing member bolt holes, (c) local deformation at the upright bolt holes (d) cross-sectional deformation of the end of the bracing members, (e) cross-sectional deformation of the uprights in the vicinity of the bolted connection, (f) axial elongation of bracing members and (g) upright overall bending and shear deformation.

309 Specifically, the contribution of the previous factors were determined by performing the following
310 changes to the FE model:

311 (a) *Bolt bending deformation*: the bending stiffness of the bolts was increased by multiplying their
312 Young's modulus E by 100 [30], effectively creating rigid bodies.

313 (b) *Local deformation at bolt hole of bracing members*: a 2 mm wide circular strip around the upright
314 holes was modelled with a high Young's modulus (multiplied by 100). This strip width was found to
315 be efficient in preventing ovalization of the holes.

316 (c) *Local deformation at bolt hole of uprights*: Similar to (b), a 2 mm wide circular strip around the
317 holes was modelled with a high Young's modulus (multiplied by 100).

318 (d) *Cross-sectional deformation of bracing members*: to prevent the cross-sectional deformation of
319 the bracing members at the bolted connections (as in Figs 19 (d) and 24 (b)), the ends of the bracing
320 members were made rigid, by increasing their Young's modulus by a factor of 100, on a length of 70
321 mm.

322 (e) *Bracing axial deformation*: the Young's modulus of the entire bracing members was increased by
323 100 to prevent their axial shortening and elongation.

324 (f) *Cross-sectional deformation of uprights*: to prevent the cross-sectional deformation of the uprights
325 at the connections, the Young's modulus of the uprights on a length of +/- 75 mm about the bolted
326 connection, was multiplied by 100.

327 (g) *Upright shear and bending deformation*: similar to (e) the Young's modulus of the entire upright
328 was multiplied by 100, so the uprights act as rigid bodies.

329 The contribution of the deformations of bracing members and uprights to the overall shear stiffness
330 of the upright frames is treated separately in this paper. The combinations of factors analysed are
331 presented in Table 4. Note that while the influence of the upright and bracing member deformations
332 are treated separately, preventing the bracing members to deform would have an influence on the
333 local deformation of the uprights. The same applies to the local behaviour of the bracing members
334 when preventing the uprights to deform.

335 Similarly to the model presented in section 4, nonlinear geometry and material analyses are performed
336 with the Young's moduli and yield stresses reported in Table 2 used as input. The influence of each
337 analysed factor on the overall shear stiffness is quantified as:

$$338 \quad \Omega = \frac{S_{modified}}{S_{initial}} \quad (5)$$

339 where $S_{initial}$ and $S_{modified}$ are the initial numerical (reported in Table 3) and modified (i.e. with
340 increased E for selected parts of the frame) shear stiffness of the upright frames, respectively,
341 calculated from Eqs (2) and (3) for the EN15512 and AS4084 test set-ups, respectively.

342 The same three upright frame configurations tested in Gilbert et al. [1] with lip-to-lip channel bracing
343 members, and reported in section 3 are used as a case study in the paper. In addition, to further analyse
344 the behaviour of the frames, the previous analyses were re-run with back-to-back channel bracing
345 members. The contribution of the aforementioned factors is analysed for these three frames for both
346 the AS4084 and EN15512 test set-ups.

347 *6.2. Lip-to-lip bracing configuration*

348 *6.2.1 Contribution of factors according to AS4084 test set-up*

349 Table 4 shows the contribution of the analysed factors on the shear stiffness of the upright frames
350 following the AS4084 test set-up and lip-to-lip bracing configuration. The values in brackets show
351 the increase in shear stiffness between two consecutive factors. For all upright frames, the effect of
352 the bolt bending on the frame shear stiffness is less than 5%. This results from the load mainly being
353 transferred in shear in the bolt from the web of the bracing members to the uprights [7, 21].

354 From Table 4, the effect of the local deformation at the bracing holes contributes the most to the shear
355 stiffness for Type A and C upright frames (about 14%). This is explained by higher contact stress at
356 bracing holes when compared to Type B upright frames due to (i) thinner bracing members used for
357 Type A upright frames and (ii) a slender cross-section for Type C uprights which is prone to cross-
358 sectional deformation, therefore leading to significant deformations of the uprights at the connections
359 and high contact forces between the bolts and other elements. This is observed in the FEA by

360 plastification occurring at the bracing holes earlier for Types A and C than for Type B upright frames.
361 The cross-sectional deformation at the ends of the bracing members is found to contribute the most
362 to the shear stiffness of the frames, especially for Type B upright frames, with the stiffness being
363 increased by more than 85%. This is attributed to Type B upright being a compact cross-section,
364 therefore not prone to cross-sectional deformation, and forcing the bracing members to deform with
365 the upright when it is twisting (Fig. 18). Consequently, local deformation is mainly concentrated to
366 the flanges of the bracing members. The axial stiffness of the bracing members was found to
367 contribute more to the overall stiffness of the frame for Type A upright frames (12%) than for Types
368 B and C.

369 Regarding the deformation of the uprights, the local deformation at the holes and the cross-sectional
370 deformation at the connections contribute more to the shear stiffness of the frame for Type C upright
371 frames than for the other two types. This is due to a slender upright cross-section and consequently
372 significant cross-sectional deformation of the uprights at the connections. Having rigid uprights
373 increases the shear stiffness by 57% and 63% for Types A and B upright frames, respectively. For
374 Type C upright frames, the upright bending stiffness contributes to the overall stiffness, about 14%.

375 *6.2.2 Contribution of factors according to EN15512 test set-up*

376 In general, when tested using the EN15512 test set-up, the analysed factors contribute to the overall
377 shear stiffness of the frames following a similar trend than when tested following the AS4084 test
378 set-up. For Type C upright frames, the axial deformation of bracing members contributes more to the
379 frame shear stiffness, about 11%, when compared to the AS4084 test set-up. For Type A upright
380 frames, the local deformation of the upright affects more the overall shear stiffness when tested
381 following the EN15512 test set-up (about 29%) than when tested following the AS4084 test set-ups
382 (about 6%). The contribution of the upright bending stiffness is also different between the two test
383 set-up for Types B and C upright frames. The differences above are attributed to the different loading
384 directions between the two test set-ups, resulting in different deformations of the frames.

385 *6.3 Contribution of factors affecting the transverse shear stiffness of back-to-back upright frames*

386 6.3.1 Contribution of factors according to AS4084 test set-up

387 Back-to-back bracing configurations typically show higher shear stiffness than lip-to-lip bracing
388 configurations [7, 15, 21]. Table 5 shows the contribution of the analysed factors on the shear
389 stiffness of the back-to-back upright frames following the AS4084 test set-up. The effect of bolt
390 bending on the shear stiffness is significant due to the back-to-back bracing pattern, now, resulting in
391 high bending moments in the bolts. This effect is less pronounced for Type A upright frames due to
392 shorter bolts being used when compared to Type B and C upright frames.

393 Similar to the lip-to-lip configurations, the effect of the local deformation at the bracing holes
394 contributes to the frame shear stiffness more for Types A and C upright frames than for Type B.
395 Cross-sectional deformation at the ends of the bracing members contributes to the shear stiffness
396 more for Type B upright frames (about 40%) than for the other two types. Unlike the lip-to-lip upright
397 frames, the axial stiffness of the bracing members is found to significantly contribute to the overall
398 stiffness of the frame, especially for Type A (141%) upright frames.

399 Regarding the deformation of the uprights and similar to lip-to-lip upright frames, effect of the local
400 deformation at the holes and cross-sectional deformation at the connections contribute more to the
401 shear stiffness, about 7% and 19% respectively, for Type C upright frames than for the other two
402 types. Upright bending stiffness contributes the most to the shear stiffness for Type A upright frames
403 (about 55%) and the least for Type C upright frames (about 25%). In overall, the upright bending
404 stiffness contributes the most to the shear stiffness of upright frames for all upright types. Compared
405 to lip-to-lip upright frames, back-to-back frames show less local deformations of the uprights at the
406 bolted connections.

407 6.3.2 Contribution of factors according to EN15512 test set-up

408 Similar to lip-to-lip upright frames, when tested using the EN15512 test set-up, the contribution of
409 the analysed factors to the overall shear stiffness of the frames presents a similar trend as when tested
410 following the AS4084 test set-up. When compared to the AS4084 test set-up, the local deformation
411 at the bolt holes of the bracing members is insignificant and about 3% for all upright frames. Unlike

412 AS4084 test set-up, the local deformation at the ends of bracing members contributes more to the
413 shear stiffness for Type A upright frames than Types B and C. The differences above are attributed
414 to different deformations of the frames due to different loading directions between the two test set-
415 ups.

416 *6.4 Improvement of the transverse shear stiffness of the upright frames*

417 The contribution of the factors affecting the most the transverse shear stiffness of the frames, and
418 reported in Tables 4 and 5, provides rack manufacturers the possibility to identify and focus on the
419 key factors to efficiently improve the upright frame performance. For the configurations analysed,
420 the following structural improvements could be performed:

- 421 • For the upright frames with lip-to-lip bracing members, as the local deformation at the end of
422 the bracing members contributes the most to the overall shear stiffness of the frames,
423 preventing this local deformation through thicker or stiffer braces would be the most efficient
424 solution to increase the shear stiffness. Increasing the second moment of area of the upright
425 through a redesigned cross-section would also impact on the value of the shear stiffness.
- 426 • For the upright frames with back-to-back bracing members, as the axial deformation of the
427 bracing members contributes the most to the overall shear stiffness of the frames, increasing
428 axial stiffness through braces with larger cross-sectional area would be the most efficient
429 solution to improve the shear stiffness of the frame. Similar to lip-to-lip upright frames,
430 thicker braces would also prevent local deformation at the end of bracing members.

431

432 **7. Conclusion**

433 In this paper, an advanced shell Finite Element model of bolted cold-formed steel storage rack upright
434 frames, with channel-bracing members, was developed for both lip-to-lip and back-to-back bracing
435 configurations. The nonlinear interaction (contact) behaviour between components at the bolted
436 connections was modelled to capture the local deformation at these locations. Results show that the
437 FE model is able to accurately capture the shear stiffness of the frames when compared to published

experimental tests, with differences ranging from 2% to 17%. The model was then used to quantify the contribution of factors influencing the transverse shear stiffness of six configurations of upright frames, including the deformation of the bolts, bracing members and uprights. Analyses were ran by deforming the frame following the testing methods in the AS4084 and EN15512 specifications. Results showed that plastification at the bolt holes starts at an early stage of loading and particularly for the frames tested following the alternative AS4084 test set-up. For lip-to-lip upright frames, (i) effect of bolt bending on the shear stiffness is insignificant and is less than 5%. (ii) local deformation at the end of the bracing members contributes the most to the overall shear stiffness of the frames, about 62% on average for both the AS4084 and EN15512 test set-ups, and (iii) effect of the upright bending stiffness on the shear stiffness is significant, about 44% on average for both the AS4084 and EN15512 test set-ups. For back-to-back upright frames, (i) bolt bending significantly influences the shear stiffness, 68% on average for the two test set-ups, (ii) unlike the lip-to-lip frames, axial deformation of bracings significantly influence the frame shear stiffness, about 81% on average for both the AS4084 and EN15512 test set-ups, and (iii) effect of upright bending stiffness on shear stiffness is significant, about 35% on average for both the AS4084 and EN15512 test set-ups.

References

1. Gilbert BP, Rasmussen KJR, Baldassino N, Cudini T, Rovere, L. Determining the transverse shear stiffness of steel storage rack upright frames. *Journal of Constructional Steel Research* 2012; 78:107-116.
2. Freitas AMS, Freitas MSR, Souza FT. Analysis of steel storage rack columns. *Journal of Constructional Steel Research* 2005; 61(8):1135-1146.
3. Bajoria KM, Sangle KK, Talicotti RS. Modal analysis of cold-formed pallet rack structures with semi-rigid connections. *Journal of Constructional Steel Research* 2010; 66(3):428-441.
4. Davies MJ. Stability of unbraced pallet racks. 5th International Speciality Conference on Cold-Formed Steel Structures. 1980; 409-428.

- 463 5. Godley M, Beale R. Investigation of the effects of looseness of bracing components in the
464 cross-aisle direction on the ultimate load-carrying capacity of pallet rack frames. Thin-walled
465 structures 2008; 46(7-9):848-854.
- 466 6. Gilbert BP, Rasmussen KJR. Determination of the base plate stiffness and strength of steel
467 storage racks. Journal of Constructional Steel Research 2011; 67(6):1031-1041.
- 468 7. Sajja S, Beale R, Godley M. Shear stiffness of pallet rack upright frames. Journal of
469 Constructional Steel Research 2008; 64(7-8):867-874.
- 470 8. Baldassino N, Bernuzzi C. Analysis and behaviour of steel storage pallet racks. Thin-Walled
471 Structures 2000; 37(4):277-304.
- 472 9. Prabha P, Marimuthu V, Saravanan M, Jayachandran, SA. Evaluation of connection flexibility
473 in cold formed steel racks. Journal of Constructional Steel Research 2010; 66(7):863-872.
- 474 10. Sajja S. Cross-aisle shear stiffness of pallet rack upright frames. 2010; Oxford Brooks
475 Univesity.
- 476 11. AS4084, Steel storage racking. 2012; Sydney, Australia: Standards Australia.
- 477 12. EN15512, Steel static storage systems – adjustable pallet racking systems – principles for
478 structural design. 2009; Brussels, Belgium: European Committee for Standardization (CEN).
- 479 13. RMI, Specification for the design, testing and utilization of industrial steel storage racks.
480 2012; Charlotte, U.S.A.: Rack Manufacturers Institute.
- 481 14. Timoshenko SP, Gere JM. Theory of Elastic Stability. Me Graw-Hill, New York, 1961.
- 482 15. Rao SS, Beale R, Godley M. Shear stiffness of pallet rack upright frames. 17th International
483 Specialty Conference on Cold-Formed Steel Structures. 2004; 295-311.
- 484 16. Bernuzzi C, Gobetti A, Gabbianelli G, Simoncelli M. Warping influence on the resistance of
485 uprights in steel storage pallet racks. Journal of Constructional Steel Research 2014; 101: 224-
486 241.
- 487 17. Bernuzzi C, Simoncelli M. EU and US design approaches for steel storage pallet racks with
488 mono-symmetric cross-section uprights. Thin-walled structures 2017; 113: 181-204.

- 489 18. Bernuzzi C, Gobetti A, Gabbianelli G, Simoncelli M. Unbraced pallet rack design in accordance
490 with European practice—Part 1: Selection of the method of analysis. *Thin-walled structures*
491 2015; 86: 185-207.
- 492 19. Bernuzzi C, Gobetti A, Gabbianelli G, Simoncelli M. Unbraced pallet rack design in accordance
493 with European practice—Part 2: Essential verification checks. *Thin-walled structures* 2015; 86:
494 208-229.
- 495 20. Sangle KK, Bajoria KM, Talicotti RS. Elastic stability analysis of cold-formed pallet rack
496 structures with semi-rigid connections. *Journal of Constructional Steel Research* 2012; 71:
497 245-262.
- 498 21. Far H, Saleh A, Firouzianhaji A. A simplified method to determine shear stiffness of thin
499 walled cold formed steel storage rack frames. *Journal of Constructional Steel Research* 2017;
500 138:799-805.
- 501 22. Roure F, Peköz T, Somalo MR, Bonada J, Pastor MM, Casafont M. Design of industrial cold-
502 formed steel rack upright frames for loads in cross-aisle direction. *International Speciality*
503 *Conference on Cold-Formed Steel Structures*. 2016; 493–507.
- 504 23. Cardoso FS, Rasmussen KJR. Finite element (FE) modelling of storage rack frames. *Journal*
505 *of Constructional Steel Research* 2016; 126:1-14.
- 506 24. Abaqus, Abaqus ver. 6.14, user manual. 2015; Providence, U.S.A.: ABAQUS, Inc.
- 507 25. Kim TS, Kuwamura H. Finite element modeling of bolted connections in thin-walled stainless
508 steel plates under static shear. *Thin-Walled Structures* 2007; 45(4):407-421.
- 509 26. Barth K, Orbison J, Nukala R. Behavior of steel tension members subjected to uniaxial
510 loading. *Journal of Constructional Steel Research* 2002; 58(5-8):1103-1120.
- 511 27. Lim JB, Nethercot D. Stiffness prediction for bolted moment-connections between cold-
512 formed steel members. *Journal of Constructional Steel Research* 2004; 60(1):85-107.

- 513 28. Chung K, Ip K. Finite element modeling of bolted connections between cold-formed steel
514 strips and hot rolled steel plates under static shear loading. *Engineering Structures* 2000;
515 22(10):1271-1284.
- 516 29. AS/NZS1252, High-strength steel bolts with associated nuts and washers for structural
517 engineering. 1996; Sydney, Australia/New Zealand Standard.
- 518 30. Yu C, Schafer BW. Simulation of cold-formed steel beams in local and distortional buckling
519 with applications to the direct strength method. *Journal of Constructional Steel Research*
520 2007; 63(5):581-590.

521 **Table 1**
522 Cross-sectional properties of the uprights and bracing members

Member	Gross area (mm ²)	I _{major axis} (mm ⁴)	I _{minor axis} (mm ⁴)	J (mm ⁴)	I _{warping} (mm ⁶)
Upright Type A	484.0	4.34×10^5	1.91×10^5	491.9	2.21×10^8
Upright Type B	674.0	1.34×10^6	2.60×10^5	802.2	6.00×10^8
Upright Type C	784.6	2.01×10^6	7.73×10^5	991.6	5.80×10^9
C35 × 20 × 1.2	100.0	1.83×10^4	4.60×10^3	47.5	1.79×10^6
C35 × 35 × 1.5	168.6	3.49×10^4	2.43×10^4	125.4	8.45×10^6

523

524

525 **Table 2**

526 Average measured material properties of uprights and bracing members

Member	E (Mpa)	F _y (Mpa)	F _u (Mpa)
Upright-Type A	202252	375	435
Upright-Type B	217254	452	499
Upright-Type C	197116	360	434
C35 × 20 × 1.2	212656	356	393
C35 × 35 × 1.5	200218	441	475

527

528

529 **Table 3**
530 Transverse shear stiffness for FEA and experimental tests

Upright type	FEA (N/mm)	Experiment ⁽¹⁾ (N/mm)	FEA/Exp
	AS4084 [11] k_{Ci}		
Type A upright frame	84	89	0.94
Type B upright frame	59.4	50.8	1.17
Type C upright frame	84.4	75.7	1.11
		Average	1.07
		CoV (%)	11.0
EN215512 [12] k_{ti}			
Type A upright frame	1.22	1.14	1.07
Type B upright frame	0.55	0.56	0.98
Type C upright frame	0.75	0.78	0.96
		Average	1.00
		CoV (%)	6.00

⁽¹⁾ Average of three tests

533
534

Table 4
Contribution of factors on shear stiffness for each structural component- lip-to-lip upright frames

Structural component	Factors	Contribution (Ω) - AS4084 test set-up ⁽¹⁾		
		Type A upright frame	Type B upright frame	Type C upright frame
Bolts	(a) Bolt bending	1.02	1.03	1.05
Bracing members	(b) Local deformation at the bolt holes	1.14	1.01	1.13
	(b) + (d) Local deformation at the end of braces	1.64 (+ 0.50)	1.86 (+ 0.85)	1.71 (+ 0.58)
	(b) + (d) + (e) Axial deformation of braces	1.76 (+ 0.12)	1.93 (+ 0.07)	1.73 (+0.02)
Uprights	(c) Local deformation at the bolt holes	1.01	1.01	1.14
	(c) + (f) Local deformation at the connections	1.07 (+ 0.06)	1.14 (+ 0.13)	1.63 (+ 0.59)
	(c) + (f) + (g) Upright bending stiffness	1.64 (+ 0.57)	1.77 (+ 0.63)	1.77 (+ 0.14)
Contribution (Ω) – EN15512 test set-up				
Bolts	(a) Bolt bending	1.02	1.02	1.04
Bracing members	(b) Local deformation at the bolt holes	1.23	1.02	1.16
	(b) + (d) Local deformation at the end of braces	1.75 (+ 0.52)	1.67 (+ 0.65)	1.75 (+ 0.59)
	(b) + (d) + (e) Axial deformation of braces	1.91 (+ 0.16)	1.73 (+ 0.06)	1.86 (+ 0.11)
Uprights	(c) Local deformation at the bolt holes	1.02	1.01	1.07
	(c) + (f) Local deformation at the connections	1.31 (+ 0.29)	1.09 (+ 0.08)	1.56 (+ 0.49)
	(c) + (f) + (g) Upright bending stiffness	1.85 (+ 0.54)	1.45 (+ 0.36)	1.92 (+ 0.36)

⁽¹⁾ The values in brackets show the increase in shear stiffness between two consecutive factors.

535
536

537
538

Table 5
Contribution of factors on shear stiffness for each structural component – back-to-back upright frames

Structural component	Factors	Contribution (Ω) - AS4084 test set-up ⁽¹⁾		
		Type A upright frame	Type B upright frame	Type C upright frame
Bolts	(a) Bolt bending	1.14	1.78	2.02
Bracing members	(b) Local deformation at the bolt holes	1.12	1.02	1.06
	(b) + (d) Local deformation at the end of braces	1.42 (+0.30)	1.42 (+0.40)	1.30 (+0.24)
	(b) + (d) + (e) Axial deformation of braces	2.81 (+1.41)	2.08 (+0.66)	1.83 (+0.53)
Uprights	(c) Local deformation at the bolt holes	1.01	1.04	1.07
	(c) + (f) Local deformation at the connections	1.15 (+0.14)	1.18 (+0.14)	1.26 (+0.19)
	(c) + (f) + (g) Upright bending stiffness	1.70 (+0.55)	1.54 (+0.36)	1.51 (+0.25)
Contribution (Ω) – EN15512 test set-up				
Bolts	(a) Bolt bending	1.19	1.89	2.03
Bracing members	(b) Local deformation at the bolt holes	1.03	1.03	1.03
	(b) + (d) Local deformation at the end of braces	1.53 (+0.50)	1.45 (+0.42)	1.36 (+0.33)
	(b) + (d) + (e) Axial deformation of braces	3.80 (+2.27)	2.12 (+0.67)	1.70 (+0.34)
Uprights	(c) Local deformation at the bolt holes	1.02	1.06	1.14
	(c) + (f) Local deformation at the connections	1.20 (+0.18)	1.32 (+0.26)	1.53 (+0.39)
	(c) + (f) + (g) Upright bending stiffness	1.64 (+0.44)	1.60 (+0.28)	1.76 (+0.23)

⁽¹⁾ The values in brackets show the increase in shear stiffness between two consecutive factors.

539
540

Fig. 1. Elements of a typical rack structure [1]

Fig. 2. Typical upright frame

Fig. 3. Upright frame shear stiffness test set-up EN15512 [12]

Fig. 4. Alternative test set up AS4084 [11]

Fig. 5. Z-pattern bracing configuration with channel bracing members ($d = 948.9 \text{ mm}$, 953.1 mm and 933.5 mm for Types A, B and C upright frames, respectively)

553
554

Fig. 6. Main dimensions of the uprights (dimensions in mm)

(a)

(b)

(c)

Fig. 7. Test set-up performed for all types of the upright frames following the alternative method in AS4084 [11] and Type A upright frame following the EN15512 [12] (shown for Type B upright frame in the AS4084) (a) overall view of the upright frame (b) vertical restraint during test (c) schematic view of the vertical restraints

(a)

(b)

(c)

Fig. 8. Test set-up performed following the method in EN15512 [12] for Type B and C upright frames (shown for Type B upright frame) (a) overall view of the upright frame (b) schematic view of the vertical restraints along the loaded upright (c) schematic view of the vertical restraints at the free end of the pinned upright

Fig. 9. Vertical restraint used in FE model for the upright frames tested following AS4084 [11] and Type A upright frame tested following EN15512 [12]

Fig. 10. Vertical restraint used in FE model for Types B and C upright frames tested following EN15512 [12] (a) Vertical restraints at four locations along the loaded upright (b) Vertical restraint at free end of the pinned upright

Fig. 11. Interactions between elements in bolted connection (master surface shown in red and slave surface shown in purple)

Fig. 12. True stress-strain curves used for all uprights and bracing members

Fig. 13. FEA and experimental load-deflection curves for AS4084 [11] test set-up, Type A upright frame

Fig. 14. FEA and experimental load-deflection curves for AS4084 [11] test set-up, Type B upright frame

Fig. 15. FEA and experimental load-deflection curves for AS4084 [11] test set-up, Type C upright frame

Fig. 16. FEA and experimental load-deflection curves for all LVDTs and AS4084 [11] test set up -Type A upright frame

(a)

(b)

Fig. 17. Deformed shapes following the AS4084 [11] test set-up, Type A upright frame, (a) FEA (with deformed scale factor of 1.0 and at a displacement of 70 mm) and (b) experimental observations

609
610
611

612
613
614
615
616

(a) (b)

Fig. 18. Deformed shapes following the AS4084 [11] test set-up, Type B upright frame, (a) FEA (with deformed scale factor of 1.0 and at a displacement of 70 mm) and (b) experimental observations

617
618

619

620

(a)

(b)

621

622

(c)

(d)

623

624

625

626

627

628

Fig. 19. Deformed shapes following the AS4084 [11] test set-up, Type C upright frame, (a) FEA overall deformation of the upright frame (with deformed scale factor of 1.0 and at a displacement of 70 mm) (b) Overall deformation of the upright frame – experimental observations (c) local deformation of the upright at bolted connection – FEA (d) local deformation of the upright at bolted connection – experimental observations

Fig. 20. FEA and experimental load-deflection curves for EN15512 [12] test set-up, Type A upright frame

Fig. 21. FEA and experimental load-deflection curves for EN15512 [12] test set-up, Type B upright frame

Fig. 22. FEA and experimental load-deflection curves for EN15512 [12] test set-up, Type C upright frame

(a)

(b)

Fig. 23. Deformed shapes following the EN15512 [12] test set-up, Type A upright frame (a) FEA (with deformed scale factor of 1.0 and at a displacement of 14 mm) and (b) experimental observation

(a)

(b)

Fig. 24. EN15512 [12] test set-up deformed shapes, Type B upright frame (a) FEA (with deformed scale factor of 1.0 and at a displacement of 14 mm) and (b) experimental observation

(a)

(b)

Fig. 25. EN15512 [12] test set-up deformed shapes, Type C upright frame (a) FEA (with deformed scale factor of 1.0 and at a displacement of 14 mm) and (b) experimental observation