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Performance-Based Seismic Design and Assessment of Multi-Storey CFS Strap-Braced Frames

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Abstract

Cold-formed steel (CFS) structures typically rely on diagonally braced stud walls to withstand lateral forces. While the response of CFS single-wall panels has been extensively investigated, limited studies focused on the seismic performance of multi-storey CFS strap-braced frames. Previous research highlighted that the presence of vertical loading can significantly reduce the lateral load and ductility capacity of strap-braced walls by amplifying the secondary moments due to P- Δ effects. While this effect is generally ignored in current design practice, it may lead to premature failures and poor seismic performance. This study aims to investigate the efficiency of a new design methodology to take into account the vertical load effects on the performance of multi-storey CFS strap-braced frames. Detailed experimentally validated FE models of CFS panels were developed in ABAQUS and used to obtain equivalent hysteretic models in OpenSees. The seismic performance of 6-storey strap-braced frames designed based on the Eurocode 8 and the proposed design methodology were then investigated under a set of artificial spectrum-compatible records. While the code-base design did not satisfy the ASCE/SEI 41-17 Life Safety (LS) and Collapse Prevention (CP) ductility limits, all performance targets were met using the proposed design methodology. The results of Incremental Dynamic Analyses (IDA) also indicated that meeting the code capacity requirements could not prevent extensive global damage in the strap-braced frames, due to soft-storey failure modes associated with the premature buckling of chord studs. Finally, the

27 efficiency of the proposed design compared to its code-compliant counterpart was
28 demonstrated under a set of 20 real spectrum compatible records, showing higher ductility
29 capacity and considerably lower damage levels.

30

31 **Keywords:** Cold-formed Steel (CFS); Strap-Braced Frames; Ductility; Performance-Based
32 Design; Incremental Dynamic Analysis (IDA); Global Damage Index.

33

34 **1 Introduction**

35 Cold-Formed Steel (CFS) structural systems are increasingly used in modern construction
36 practices due to their unique advantages, such as lightweight, flexibility in cross-sectional
37 shapes, and ease of handling and transportation compared to conventional hot-rolled steel
38 structures. In conventional multi-storey CFS buildings, strap-braced stud walls are widely used
39 as one of the primary force-resisting system against lateral wind and earthquake loads. The
40 lateral resistance is generally provided by diagonal X-shaped bracing members [1], while knee-
41 braced [2], K-braced [3–5], or a combination of K- and X-shaped braces [6] have also been
42 utilised. Previous studies indicated that the structural response of the strap-braced stud walls
43 can be considerably affected by the wall's aspect ratio [7–9]. It was shown that ratios greater
44 than 1:2 might lead to a less ductile behaviour due to the premature failure of the chord studs.
45 Studies on the performance of walls employing welded joints [8,9] suggested a ductile
46 response, provided a careful design and fabrication are implemented. In common practice, CFS
47 strap-braced walls are sheathed with a wide range of materials like gypsum, OSB or plywood
48 boards. Although they can increase the lateral load capacity through their stiffness [10], a
49 composite action with the diagonal braces is not accounted for in the current design codes as a
50 result of their brittle nature [11].

51 Under seismic load actions, diagonally strap braced systems dissipate energy mainly by
52 the plastic deformation of the tensile straps, as the straps in compression buckle at a very early
53 loading stage due to their high width-to-thickness ratio (or high slenderness). Although the
54 straps usually have a uniform cross-section over their total length, it has proven beneficial to
55 reduce the width over a length in the middle, acting as a seismic fuse and protecting the
56 susceptible elements such as the connections of the straps with the chord studs [1]. Currently,
57 the primary approach to avoid brittle modes of failure in CFS stud wall systems is achieved by
58 implementing capacity design rules. The aim is to provide non-dissipative members with a
59 desirable level of over-strength, to sustain the maximum forces anticipated in the plastic
60 regions. This eventually allows the system to develop ductility (i.e. the ability to experience
61 large plastic deformations while maintaining their yield capacity) through yielding at controlled
62 locations [12,13]. The highest ductility capacity can be attained if the braces have sufficient
63 ductility [14–16] and premature failure modes in the chord studs are prevented before the
64 diagonal straps reach their ultimate strain [17].

65 There are limited experimental research studies on the seismic response of CFS multi-
66 storey systems employing diagonal braces. Aiming to study the non-linear dynamic behaviour
67 of CFS structural systems, Kim et al. [18] performed full-scale shaking table tests on two-
68 storey one-bay strap-braced frames with box-shaped chord studs and welded connections. A
69 maximum drift ratio of 3% at the first storey was reported, and no obvious damage was
70 observed, confirming the suitability of the system for seismic applications. In a similar study,
71 Fiorino et al. [19] tested three-storey two-bay CFS strap-braced wall frames designed per
72 Eurocode 8 as non-dissipative systems. The system exhibited satisfactory performance and
73 achieved a maximum drift ratio of over 2%, without significant damage.

74 In their numerical study at the structural level, Lee and Foutch [20] performed a modified
75 incremental dynamic analysis of two-, four- and six-storey strap-braced wall frames,

76 employing twenty accelerograms. They concluded that the R factor of 4 was adequate for this
77 system, while the per FEMA 355F [21] estimated collapse drift capacities were too
78 conservative. Velchev [1] and Comeau et al. [22] assessed the AISI S213 Canadian and FEMA
79 P695 seismic force modification factors, alongside the building height limits for multi-storey
80 concentrically strap-braced systems. Their studies, in general, confirmed the code suggested
81 values for limited ductility (LD) and conventional construction (CC) systems. In another
82 relevant study, Fiorino et al. [23] conducted a numerical study following FEMA P695 [24]
83 specifications to assess the behaviour factor for CFS strap-braced stud wall systems. They
84 investigated the seismic performance of a set of fourteen archetypes using CFS strap-braced
85 stud walls featuring screwed connections with gusset plates. The results indicated the suitability
86 of the system for seismic applications; however, the studied structures exhibited a relatively
87 low behaviour factor (q) around 2.5. Davani et al. [25] also evaluated the performance of nine
88 full-scale wall specimens based on the experimental work of Moghimi and Ronagh [11],
89 subjected to 14 ground motions. Their study showed that the contribution of cladding and
90 corner brackets can increase the lateral strength and stiffness and reduce the occurrence of
91 damage in strap-braced wall systems.

92 Experimental test studies on the monotonic and cyclic response of single-wall panels were
93 generally conducted without accounting for any additional vertical loads. Additional vertical
94 loads have been applied only in a few experimental studies focusing on sheathed walls without
95 employing diagonal straps [26–28]. However, due to their inherently different mechanism, the
96 results do not apply to strap-braced systems. It should be noted that Eurocode 8 [29] does not
97 contain explicit design rules for CFS stud wall systems, and the ones intended for conventional
98 hot-rolled steel structures are generally used. Following these, the lateral load capacity of strap-
99 braced walls is based on the tensile strength of the straps alone, under zero vertical loading.

100 Therefore, the secondary bending moments due to $P-\Delta$ effects are not currently accounted for

101 in the seismic design of CFS strap-braced stud wall systems. AISI S400-20 [30] accounts for
102 the bending moment due to P- Δ effects, but for walls with a height-to-width ratio greater than
103 1.9:1. However, research recently conducted by Papargyriou et al. [17] suggested that the
104 lateral load and ductility capacities of strap-braced wall systems were adversely affected by
105 additional vertical loads leading to premature failure modes in the chord studs due to P- Δ
106 effects, even for 1:1 ratios.

107 In a recent study by Papargyriou and Hajirasouliha [31], a new methodology was
108 developed for designing CFS strap-braced wall frames, proposing preliminary design formulae
109 to predict the lateral load and ductility capacity of strap-braced stud walls, considering different
110 cross-sectional thicknesses and vertical load levels. The efficiency of the proposed method was
111 demonstrated by designing a CFS 6-storey frame following Eurocodes' guidelines. It was
112 shown that although the member capacity checks were satisfied, the ductility of the system was
113 well below the target ductility at some storey levels, yielding a brittle response and an
114 unacceptable seismic performance. By applying their proposed methodology, however, the
115 design solution could satisfy both the Eurocode capacity checks and the target ductility
116 demands at different earthquake intensity levels.

117 The present work aims to assess the efficiency of the design methodology proposed by
118 Papargyriou and Hajirasouliha [31], for the first time, at the structural level and provide a better
119 understanding of the seismic performance of CFS strap-braced multi-storey systems by
120 quantifying their global damage and failure mechanism at different earthquake intensity levels.
121 Detailed non-linear FE models of CFS panels are developed in ABAQUS [32], and their
122 accuracy is demonstrated against experimental results in the literature. To increase the
123 computational efficiency, the validated models are then used to obtain equivalent hysteretic
124 models in OpenSees [33]. The seismic performance of 6-storey strap-braced frames designed
125 based on the Eurocode-8 and the proposed design methodology is then investigated under a set

126 of artificial spectrum-compatible records under increasing intensity (i.e. incremental dynamic
127 analyses) as well as twenty real earthquake ground motion records representing the selected
128 design spectrum. Finally, the efficiency and reliability of the design solutions are assessed
129 based on ASCE/SEI 41-17 ductility limits for different performance levels, wall ductility
130 capacities obtained from the validated FE models in ABAQUS, and a global cumulative
131 damage index. The results indicated that the Eurocode design solution, ignoring the effect of
132 vertical loads in the ductility capacity, did not fulfil the Life Safety (LS) and Collapse
133 Prevention (CP) performance levels and sustained extensive damage even at low earthquake
134 intensity levels. However, the proposed methodology yielded improved seismic performance
135 with higher ductility capacity and significantly lower damage levels.

136 **2 Proposed design methodology**

137 In a previous study by Papargyriou et al. [17], it was found that additional vertical loads
138 imposed to a CFS strap-braced stud wall led to increased secondary moments in the
139 compressive chord stud due to P- Δ effects. The interaction of the axial loads and the secondary
140 moments can result in the premature buckling failure of chord studs before straps reach their
141 ultimate strain and subsequently reduce the lateral load-bearing capacity and ductility of the
142 system. To address this issue, Papargyriou and Hajirasouliha [31] proposed a design
143 methodology to estimate the lateral load-bearing and ductility capacity of CFS strap-braced
144 stud walls under the presence of vertical loads. Their proposed procedure can be summarised
145 as follows:

- 146 • Step 1: The lateral response of a reference CFS strap-braced stud wall (wall 1) under
147 monotonic loading is obtained using a detailed FE model or experimental test results. No
148 vertical load is applied at this stage, and therefore the failure mode is expected to be due
149 to gross cross-section rupture of the straps.

150 • Step 2: Under zero vertical loading, Eq. (1) is used to calculate the lateral load capacity
 151 ($F_{u,2}$) of a selected CFS strap-braced stud wall (wall 2) with the same configuration and
 152 material properties as wall 1, but different element sizes. It is related to the lateral load
 153 capacity of wall 1 ($F_{u,1}$), calculated in the previous step, as a function of the diagonal strap
 154 thicknesses ($t_{s,1}$) and ($t_{s,2}$) and chord stud thicknesses ($t_{ch,1}$) and ($t_{ch,2}$) of walls 1 and 2,
 155 respectively.

$$F_{u,2} = F_{u,1} \cdot f1 \left(\frac{t_{s,2}}{t_{s,1}}, \frac{t_{ch,2}}{t_{ch,1}} \right) \quad (1)$$

156 • Step 3: The ductility of reference wall 1 (μ_1), calculated in step 1, is used to estimate the
 157 ductility of wall 2 (μ_2) under the condition of zero vertical loading, with the aid of Eq. (2).

$$\mu_2 = \mu_1 \cdot f2 \left(\frac{t_{s,2}}{t_{so,2}} \right) \quad (2)$$

158 In the above equation ($t_{so,2}$) is the diagonal strap thickness of wall 2, which leads to the
 159 simultaneous failure of the strap and the chord stud. A simple design process is proposed
 160 to obtain ($t_{so,2}$) using the axial-bending moment interaction relationship
 161 $F_c/N_{c,Rd} + M/M_{c,Rd} = 1$, where (F_c) is the compressive component of the strap force
 162 transferred on the chord stud, ($N_{c,Rd}$) and ($M_{c,Rd}$) are the compressive and bending moment
 163 capacities of the chord stud, respectively, calculated in accordance with EN 1993 [34,35],
 164 and (M) is the secondary bending moment due to P- Δ effects.

165 • Step 4: At this stage, Eqs. (3) and (4) are used to predict the lateral load capacity ($F_{u,P,2}$)
 166 and ductility ($\mu_{P,2}$) of wall 2, under the presence of a vertical load (P), expressed as a ratio
 167 against the total vertical load capacity of the wall ($P_{w,2}$), obtained by adding the axial load
 168 capacities of the intermediate and chord studs calculated per Eurocode 3 [34,35].

$$F_{u,P,2} = F_{u,2} \cdot f3 \left(\frac{P}{P_{w,2}} \right) \quad (3)$$

$$\mu_{P,2} = \mu_2 \cdot f4 \left(\frac{P}{P_{w,2}} \right) \quad (4)$$

169 The functions f_1 , f_2 , f_3 and f_4 were obtained based on the results of a comprehensive
 170 parametric study conducted by Papargyriou and Hajirasouliha [31] on a wide range of CFS
 171 strap-braced stud walls. Table 1 lists the adopted functions for the wall configuration used in
 172 this study (see Fig. 1). It should be noted that while the proposed design methodology is general,
 173 these functions may slightly change if a very different bracing system is utilised.

174

175 **Table 1** Functions f_1 , f_2 , f_3 and f_4 used to obtain lateral load capacity and ductility per Eqs. (1), (2),
 176 (3) and (4) (adopted from Papargyriou and Hajirasouliha [31])

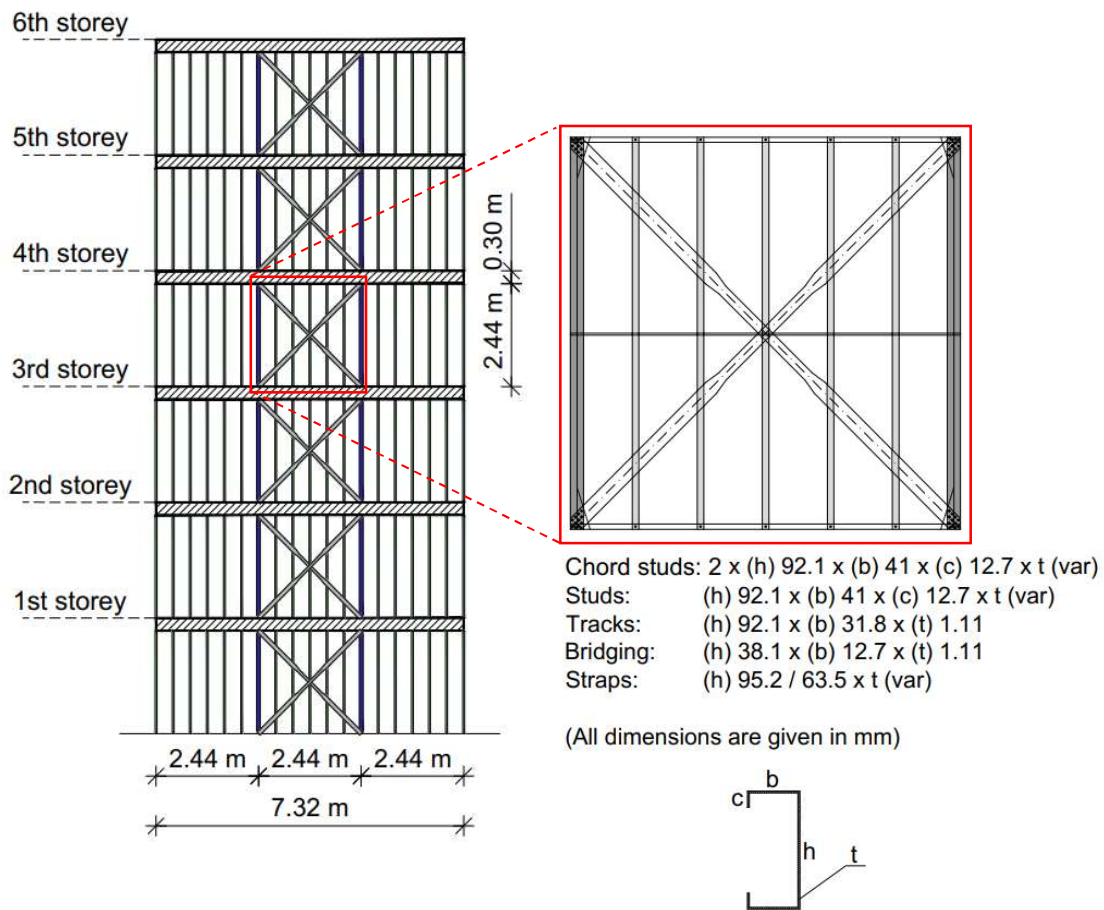
$f_1 = \frac{F_{u,2}}{F_{u,1}} = 0.85 \left(\frac{t_{s,2}}{t_{s,1}} \right) + 0.15 \left(\frac{t_{ch,2}}{t_{ch,1}} \right)$
$\begin{cases} f_2 = \frac{\mu_2}{\mu_1} = 1, & t_{s,2} \leq t_{so,2} \\ f_2 = \frac{\mu_2}{\mu_1} = -0.09 + 2.16 \left(\frac{t_{s,2}}{t_{so,2}} \right) - 1.07 \left(\frac{t_{s,2}}{t_{so,2}} \right)^2, & t_{s,2} > t_{so,2} \end{cases}$
$f_3 = \frac{F_{u,P,2}}{F_{u,2}} = 1 - 0.60 \left(\frac{P}{P_{w,2}} \right) + 0.30 \left(\frac{P}{P_{w,2}} \right)^2$
$f_4 = \frac{\mu_{P,2}}{\mu_2} = A \left(\frac{P}{P_{w,2}} \right)^3 + B \left(\frac{P}{P_{w,2}} \right)^2 + C \frac{P}{P_{w,2}} + 1$ $A = 10.47 \left(\frac{t_{s,2}}{t_{so,2}} \right)^3 - 44.19 \left(\frac{t_{s,2}}{t_{so,2}} \right)^2 + 30.69 \left(\frac{t_{s,2}}{t_{so,2}} \right) + 4.85$ $B = -8.55 \left(\frac{t_{s,2}}{t_{so,2}} \right)^3 + 31.65 \left(\frac{t_{s,2}}{t_{so,2}} \right)^2 - 18.64 \left(\frac{t_{s,2}}{t_{so,2}} \right) - 7.47$ $C = -1.75 \left(\frac{t_{s,2}}{t_{so,2}} \right)^3 - 5.95 \left(\frac{t_{s,2}}{t_{so,2}} \right)^2 + 3.05 \left(\frac{t_{s,2}}{t_{so,2}} \right) + 0.81$

177

178 2.1 Design solutions of a case study frame

179 To demonstrate the efficiency of the proposed methodology, a case study multi-storey CFS
 180 frame was designed following Eurocodes' specifications. The frame had six storeys above
 181 ground level, each having a height of 2.74 m, and three 2.44 m span bays. The middle span at

182 each level was configured as the strap-braced wall panel, providing the lateral bearing
 183 resistance (see Fig. 1). The cross-sectional dimensions of the walls' elements, including the
 184 studs, the top and bottom tracks, the bridging element, and the diagonal straps, are shown in
 185 Fig. 1. The chord studs consisted of two back-to-back single lipped-channel sections, while the
 186 diagonal straps had a "dogbone" shape, having a reduced depth over a length of 762 mm in the
 187 middle. The thicknesses of the studs and the diagonal straps varied along the frame height. A
 188 tributary width of 5.0 m was assumed for the calculation of the gravity loads and the storey
 189 masses.



190
 191 **Fig. 1** Geometry of the case-study strap-braced wall frame
 192

193 **2.2** *Design specifications*

194 The frame was assumed to be part of a residential-use building, of importance class II,
 195 designed for a permanent load $g = 1.5 \text{ kN/m}^2$ and a live load $q = 2 \text{ kN/m}^2$, while the partial

196 safety factors per EN 1990 [36] were $\gamma_G=1.35$ and $\gamma_Q=1.50$, respectively. The storeys all had
197 equal masses, calculated from the combination of the seismic action in accordance with
198 EN 1990 [36]: $G_k + \psi_{E,i} \cdot Q_k$, where (G_k) and (Q_k) represent the permanent and variable (live)
199 loads, respectively, and ($\psi_{E,i}$) is the coefficient for variable actions taken as 0.3. The site was
200 assumed to be in a high seismicity area, with a reference peak ground acceleration $\alpha_{gR}=0.35$ g
201 and a ground type B (i.e. deposits of very dense sand, gravel, or very stiff clay). The building
202 was assumed to be regular in plan and elevation, and the base shear force was calculated and
203 distributed along the height following the lateral force method per Eurocode 8 [29].

204 2.3 *Conventional and proposed design solutions*

205 For the conventional design solution, the frame members were designed in accordance
206 with Eurocode 3 [34,35] and the capacity design rules of Eurocode 8 [29]. However, as
207 discussed before, the effects of vertical loads on the lateral load capacity and ductility of the
208 strap-braced wall frames are not taken into account in the Eurocode design process. To address
209 and further study this issue, the same frame was designed following the proposed methodology,
210 explained in section 2 (proposed design solution). The thicknesses of the studs (t_{ch}) and the
211 diagonal straps (t_s) of the Eurocode and the proposed design solutions are listed in Table 2,
212 while the general dimensions of the sections are shown in Fig. 1.

213 Following an eigenvalue analysis, the fundamental periods of the Eurocode and the
214 proposed design solutions were 0.82 sec and 0.78 sec, respectively. It should be noted that the
215 period used for the initial design of the frames was 0.408 sec and calculated based on EC8
216 proposed design equation.

217 **Table 2** The plate thickness of the members in the Eurocode and the proposed design solutions (sizes
 218 are shown in Fig. 1)
 219

Storey No	Eurocode design		Proposed design	
	Stud thickness t_{ch} (mm)	Diagonal strap thickness t_s (mm)	Stud thickness t_{ch} (mm)	Diagonal strap thickness t_s (mm)
1	2.0	4.0	2.0	4.0
2	1.5	4.0	2.0	4.0
3	1.5	4.0	2.0	4.0
4	1.16	3.0	1.5	3.0
5	1.16	3.0	1.5	3.0
6	1.16	1.5	1.16	1.5

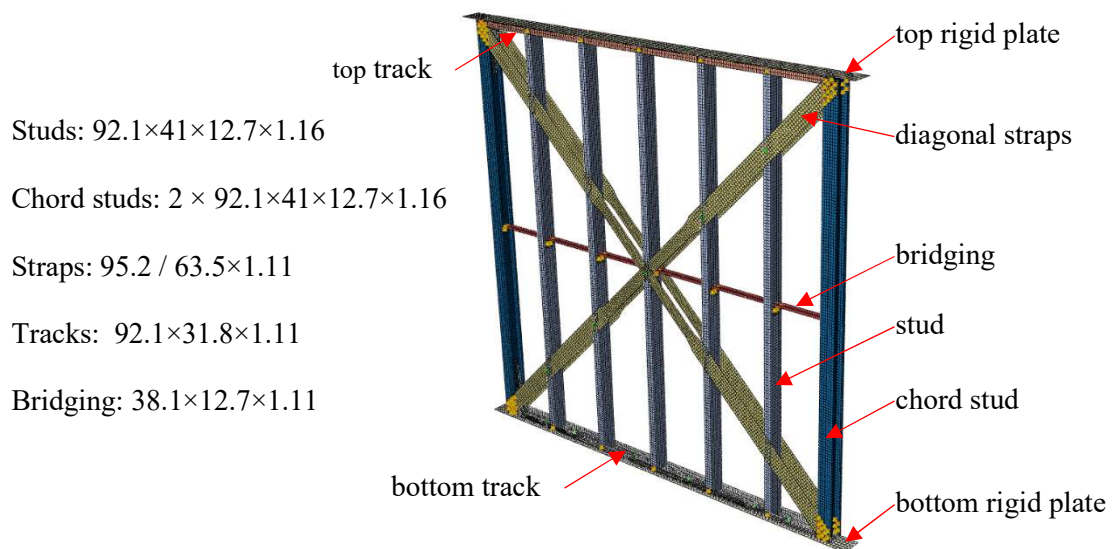
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221 3 Development of non-linear numerical models

222 Numerical modelling has been widely used as a cost-effective and efficient means to
 223 predict the response of CFS elements and structural systems under different loading conditions
 224 [5,37]. In this study, ABAQUS software [32] was utilised to simulate the non-linear behaviour
 225 of CFS strap-braced wall panels, as previous studies showed that it can provide accurate results
 226 for thin-walled CFS elements and connections [38–41]. The developed models were validated
 227 against available experimental data and were then used to: a) derive the load-displacement
 228 curves for the wall panels of the Eurocode and the proposed design solutions, as discussed in
 229 section 2, and b) obtain equivalent non-linear hysteretic models in OpenSees software [33] to
 230 simulate the non-linear seismic behaviour of multi-storey CFS strap-braced frames under
 231 different earthquake intensity levels. Using this approach can significantly reduce the
 232 computational costs of non-linear dynamic analyses of such complex systems. However, it
 233 should be noted that, in general, the wall panels have a different lateral response at each storey,
 234 depending on their cross-sectional dimensions and the amount of vertical loading, and therefore
 235 equivalent models should be obtained for them individually.

236 3.1 Detailed non-linear numerical model in ABAQUS

237 As shown in Fig. 2, the detailed wall models in ABAQUS comprised the chord and
 238 intermediate studs, the top and bottom tracks, and the bridging element at the mid-height of the
 239 wall. Two pairs of diagonal dogbone-shaped straps were used as bracing elements. The element
 240 dimensions are shown in Fig. 2. The model accounted for material non-linearity and P- Δ effects.
 241 The initial geometric imperfections were omitted in this work since previous studies [17]
 242 showed that they had a negligible effect on the lateral response of the system. **Table 3**
 243 summarises the engineering values of yield (f_y) and ultimate (f_u) stresses for the structural
 244 elements. The modulus of elasticity (E) and the Poisson's ratio (ν) had values of 203 GPa and
 245 0.3, respectively.



246

247 **Fig. 2** Detailed ABAQUS numerical model

248 **Table 3** Material properties of ABAQUS model

Element	Yield stress f_y (MPa)	Ultimate stress f_u (MPa)
Straps	296	366
Studs / Chord studs	325	382
Tracks	296	366

249

250 In the following section, the key features of the developed FE model are presented. For a
251 more thorough description and discussion, the reader can refer to [17].

252

253 3.1.1 *Summary of model features*

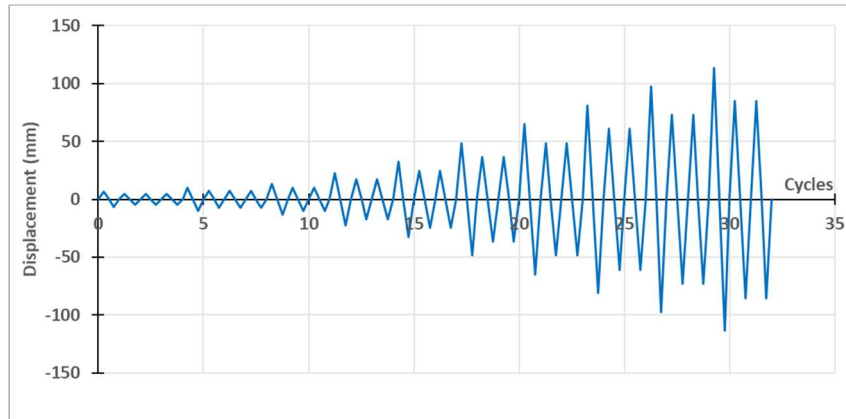
254 Similar to the reference experimental test set-up, the developed models featured the hold-
255 down devices and anchor rods at the chord stud top and bottom ends alongside the connections
256 of the tracks through shear anchors. Over the top and below the bottom track, rigid plates were
257 used to transfer the vertical loads uniformly to the chord studs, the intermediate studs and the
258 foundation. The bridging element, intended to reduce the effective length of the vertical
259 elements, was attached to them through L-shaped brackets.

260 The ABAQUS “S4R” shell element (four-node element with three translational and three
261 rotational degrees of freedom per node) was used to discretise the wall’s elements as it was
262 proven accurate in predicting the behaviour of CFS systems in the previous studies [38,39,41].
263 Following a mesh sensitivity analysis, the lateral load capacity value converged at a mesh size
264 of 15 mm × 15 mm, yielding a difference of less than 0.5% compared to a coarser mesh of
265 20 mm × 20 mm.

266 The straps were connected to the chord studs and the top and bottom tracks through 12 No
267 10 (Ø10 mm) self-drilling screws, while the intermediate studs were connected to the top and
268 bottom tracks through 1 No 8 (Ø8 mm) self-drilling screw per side, at each end. The screwed
269 connections were modelled using discrete fasteners, which were assigned connector properties,
270 defining the relative motion between the connected surfaces. The proposed equations in Pham
271 and Moen’s work [42], derived based on their experimental work on steel screw-fastened
272 connections, were adopted to describe the strength and stiffness of the connectors.

273 The vertical load was applied in the first analysis step as a uniformly distributed surface
274 load on the rigid top plate, and the lateral displacements were imposed in the second step. A

275 maximum displacement value of 300 mm was applied for monotonic analysis, whereas cyclic
276 loading followed the reversed cyclic protocol suggested by ASTM E2126 [43] , as shown in
277 Fig. 3.

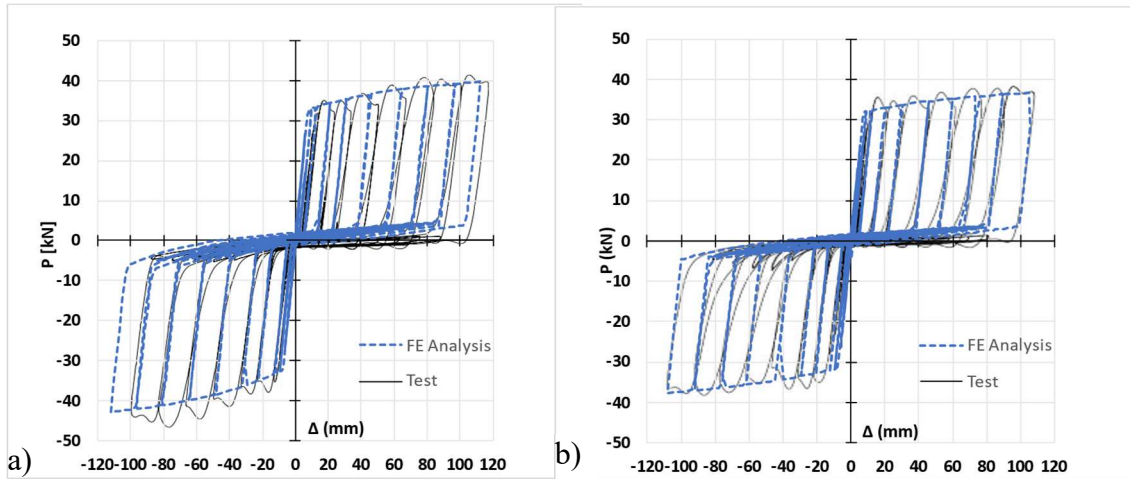


278
279 **Fig. 3** Reversed cyclic protocol per ASTM E2126 [43]
280

281 In order to transfer the vertical load and account for the correct interaction between the
282 structural elements, surface-to-surface interactions were described, using a “hard” contact
283 definition in the normal direction and a “frictionless” contact in the transverse direction.

284 3.1.2 Model validation

285 The developed FE model was verified under cyclic loading against the results of test
286 specimens 26A-C and 32A-C included in the experimental work of Velchev [1]. The lateral
287 load-displacement responses of the test and the FE analyses are compared in Fig. 4. The results,
288 in general, demonstrate the efficiency of the adopted FE models to simulate the hysteretic
289 response of the tested CFS strap-braced walls. The average error in the estimated cumulative
290 energy dissipation (i.e. the area enclosed by the load-displacement curves) of the walls was 18%
291 and 17% for the test specimens 26A-C and 32A-C, respectively. A more comprehensive
292 comparison can be found in [17].



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294
295

Fig. 4 Comparison of the experimental and FE analysis predicted cyclic responses of specimens a) 26A-C, and b) 32A-C

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To assess the efficiency of the Eurocode and the proposed design methodology in terms of the lateral load capacity and ductility of the wall panels, the validated FE models were adopted to obtain the load-displacement curves for all the wall panels of the Eurocode and the proposed design solutions by taking into account their cross-sectional dimensions and the amount of vertical loading. The results were then used to calculate the ultimate load capacity (F_u) and ductility (μ) for each wall panel. Ductility was expressed as a ratio of the ultimate displacement (Δ_u) over the displacement at yield (Δ_y) (i.e. $\mu = \Delta_u/\Delta_y$). The ultimate displacement was taken as the minimum of the displacement corresponding to the failure of the wall (due to strap rupture or buckling of the compressive chord stud) or the displacement corresponding to a 20% drop of the ultimate load capacity. To obtain the values of (Δ_u) and (Δ_y), idealised bilinear curves of the actual responses, based on the FEMA 356 [44], were used.

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308

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311

The ultimate lateral load (F_u) and ductility (μ) capacities of the walls at each storey level are summarised in Table 4 for the Eurocode and the proposed design solutions. It can be noted that, despite satisfying the code's load capacity checks, the Eurocode design solution exhibits relatively low ductility capacity at storeys 2 to 5, well below the target ductility of 4 expected for ductility class medium (DCM) structures. As discussed before, this is mainly attributed to

312 the premature buckling of the chord studs caused by the interaction of the compressive force
 313 and secondary moment due to P- Δ effects, amplified by the presence of vertical loads. By
 314 contrast, the proposed design solution reached high ductility values at all storeys.
 315

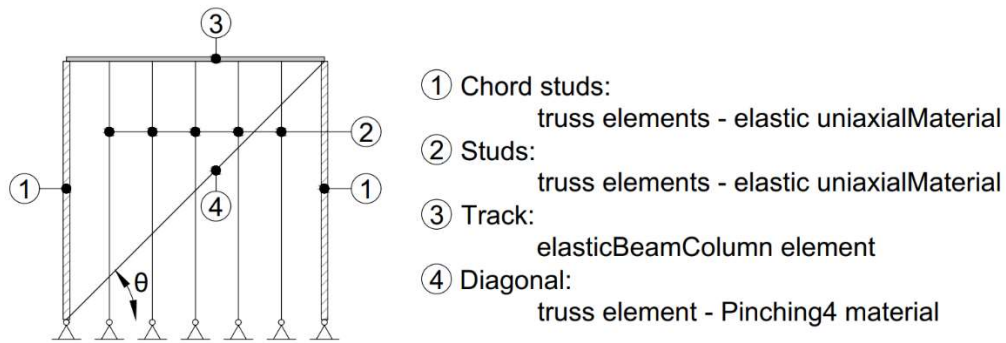
316 **Table 4** Lateral load and ductility capacities of the Eurocode and the proposed design solutions

Storey No	Eurocode design		Proposed design	
	F _u (kN)	μ	F _u (kN)	μ
1	120.4	14.8	120.3	14.8
2	105.2	1.9	121.7	15.5
3	108.8	2.1	123.1	15.9
4	80.7	2.4	93.0	15.3
5	85.5	2.6	94.4	16.3
6	49.1	21.3	49.1	21.3

317

318 3.2 *Non-linear numerical models in OpenSees*

319 In this study, OpenSees software [33] is used to assess the seismic performance of multi-
 320 storey CFS strap-braced frames. Fig. 5 shows the OpenSees numerical model of a single wall
 321 panel. It comprised the intermediate and chord studs, the top track and one diagonal element
 322 that represents the overall hysteretic behaviour of the X-shaped braces. The studs and chord
 323 studs were modelled with “truss” elements and assigned elastic “uniaxialMaterial” properties.
 324 The top track was modelled using the “elasticBeamColumn” element, which supports
 325 uniformly distributed loading and incorporates elastic material properties. The non-linear
 326 lateral behaviour of the wall was simulated by using the diagonal element with the non-linear
 327 “Pinching4” material, derived from the corresponding monotonic analysis of detailed
 328 ABAQUS numerical models for each wall panel. In practical terms, this single diagonal
 329 element controls the non-linear lateral performance of the wall, while the effects of local
 330 buckling and failure of all the elements on the overall response of the system are indirectly
 331 included. This is explained in more detail in the next section.



332

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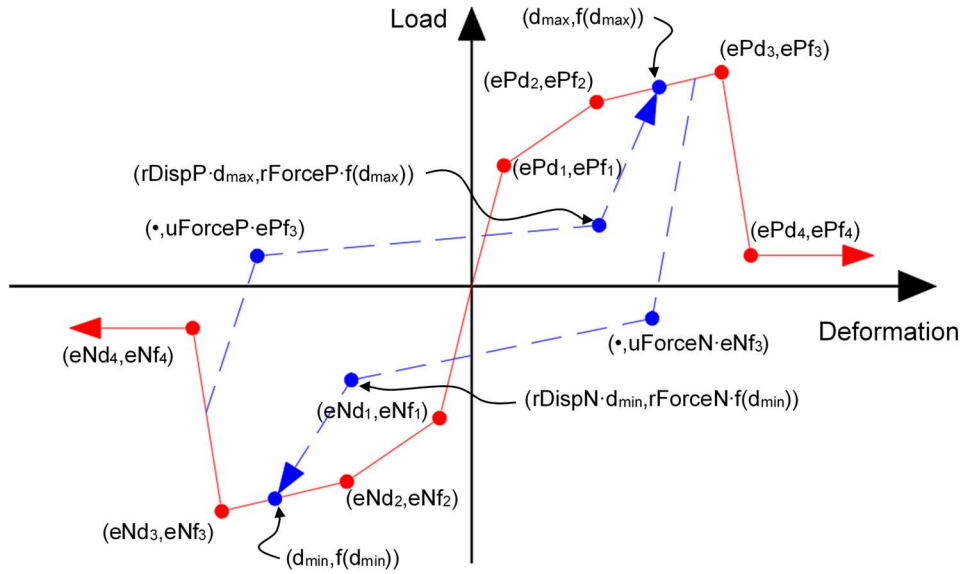
Fig. 5 Single strap-braced wall model in OpenSees

334

335 3.3 *Hysteretic response*

336 In general, strap-braced stud walls subjected to cyclic loading exhibit a pinching behaviour,
 337 which results from only the tensile straps being active in every excursion [18]. Therefore, in
 338 this study, the hysteretic response of the wall was defined by assigning the hysteretic
 339 “Pinching4” material (see Fig. 6) of OpenSees [33] to the diagonal element which in previous
 340 studies [23,45] was proven accurate in capturing this behaviour.

341 The “Pinching4” material definition comprises 39 parameters, including the backbone
 342 curve points of the cyclic response for both positive and negative loading directions, the
 343 pinching, and finally, the strength and stiffness degradation parameters. Since neither strength
 344 nor stiffness degradation was observed in the studied walls, their corresponding parameters
 345 were set equal to zero.



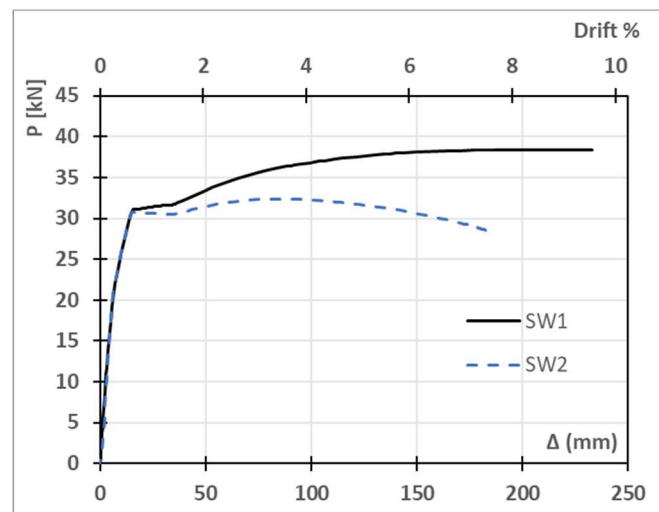
346
347 **Fig. 6** Definition of OpenSees [33] “Pinching4” hysteretic model
348

349 The point pairs (ePd_1, ePf_1) , (ePd_2, ePf_2) , (ePd_3, ePf_3) and (ePd_4, ePf_4) , defining the positive
350 loading direction backbone envelope, were selected from the bi-linearised, per FEMA 356 [44],
351 monotonic lateral load-displacement relationship of the corresponding ABAQUS numerical
352 models, whilst the pairs (eNd_1, eNf_1) , (eNd_2, eNf_2) , (eNd_3, eNf_3) and (eNd_4, eNf_4) , representing
353 the negative-loading backbone curve were set equal to their negative symmetrical ones. It
354 should be noted that since these points were assigned as material properties of the diagonal
355 element, they needed to be transformed into equivalent stresses ($\sigma = P / (A \times \cos\theta)$) and strains
356 ($\varepsilon = \Delta \times \cos \theta / L$). In the above equations (P) is the lateral force, (A) the area of one pair of straps,
357 (Δ) the lateral displacement, (L) the initial diagonal length and (θ) the angle of the diagonal
358 (see Fig. 5).

359 The pinching controlling parameters ($rDispP$, $rForceP$, $uForceP$, $rDispN$, $rForceN$, and
360 $uForceN$) were obtained through a calibration process. Their values were iteratively modified,
361 aiming to reach the best agreement between the OpenSees analysis and the actual cyclic
362 response from ABAQUS models and to minimise the difference in the cumulative dissipated
363 energies. The results indicated that, in general, these parameters are not very sensitive to the

364 imposed vertical load level. Following the calibration process, the pinching parameter values
365 of (rDispP=0.9, rForceP=0.03, uForceP=0, rDispN=0.9, rForceN=0.03, and uForceN=0) were
366 found to be suitable for all cases, and therefore, these values were used in the analyses
367 henceforth.

368 The accuracy of the equivalent OpenSees model was investigated for two cased study
369 examples. Wall SW1 had no additional vertical load and failed because the straps reached their
370 ultimate strain. Wall SW2 had an additional vertical load equal to 29% of its axial load capacity
371 (P_w), and its failure mode was the premature buckling of the compressive chord stud before the
372 straps reached their ultimate strain. Fig. 7 compares the monotonic lateral load-displacement
373 responses of these wall specimens.

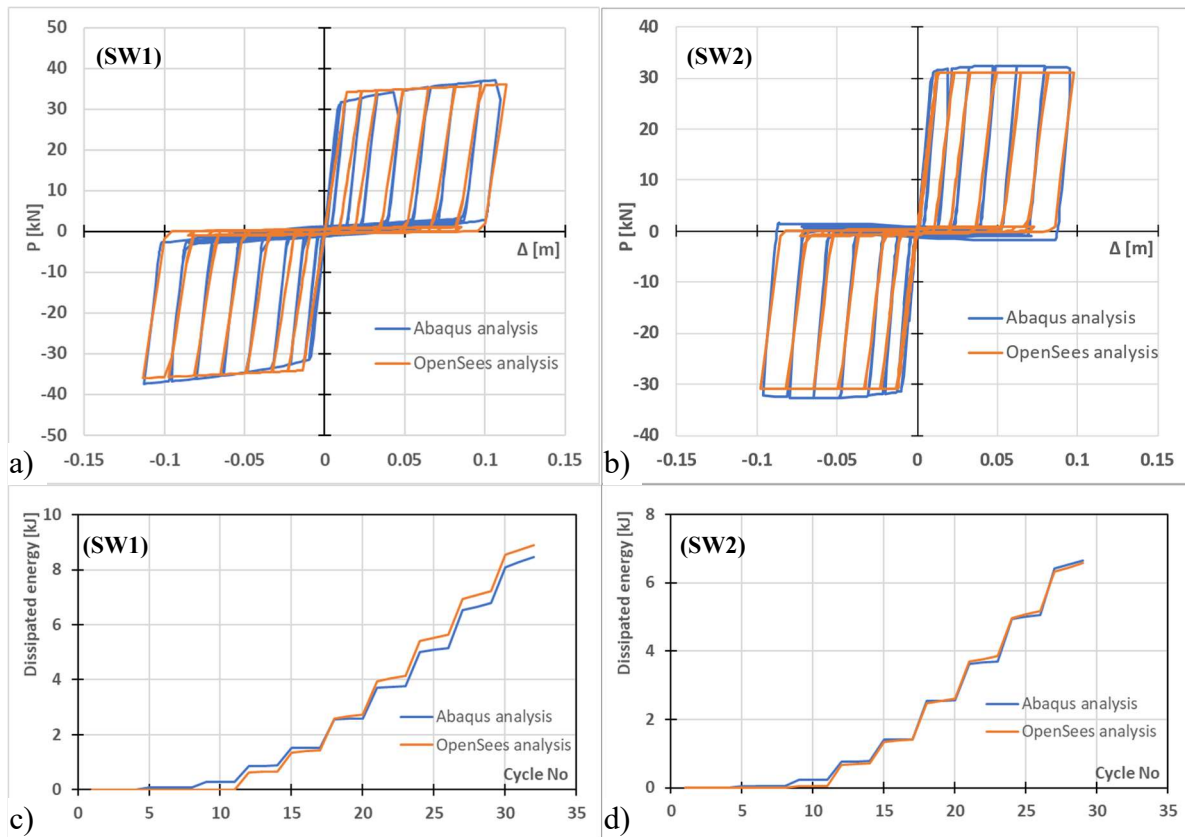


374

375 **Fig. 7** Lateral load-displacement relationships for walls SW1 and SW2

376

377 In Fig. 8, the accuracy of the “Pinching4” model is verified by comparing the hysteretic
378 response of the SW1 and SW2 models in ABAQUS and OpenSees, in terms of lateral-load
379 displacement curves and the cumulative dissipated energy. It can be observed from Fig. 8a and
380 Fig. 8b that for both cases, there was a good agreement between the hysteretic lateral load-
381 displacement curves with respect to the initial stiffness and the maximum capacity (P_u).
382 Regarding the cumulative dissipated energy, the average error was 8.6% for wall SW1 and 4.5%
383 for wall SW2 (see Fig. 8c and Fig. 8d).



384

385 **Fig. 8** ABAQUS and OpenSees hysteretic response and cumulative energy of (a & c): SW1 wall and
 386 (b & d): SW2 wall

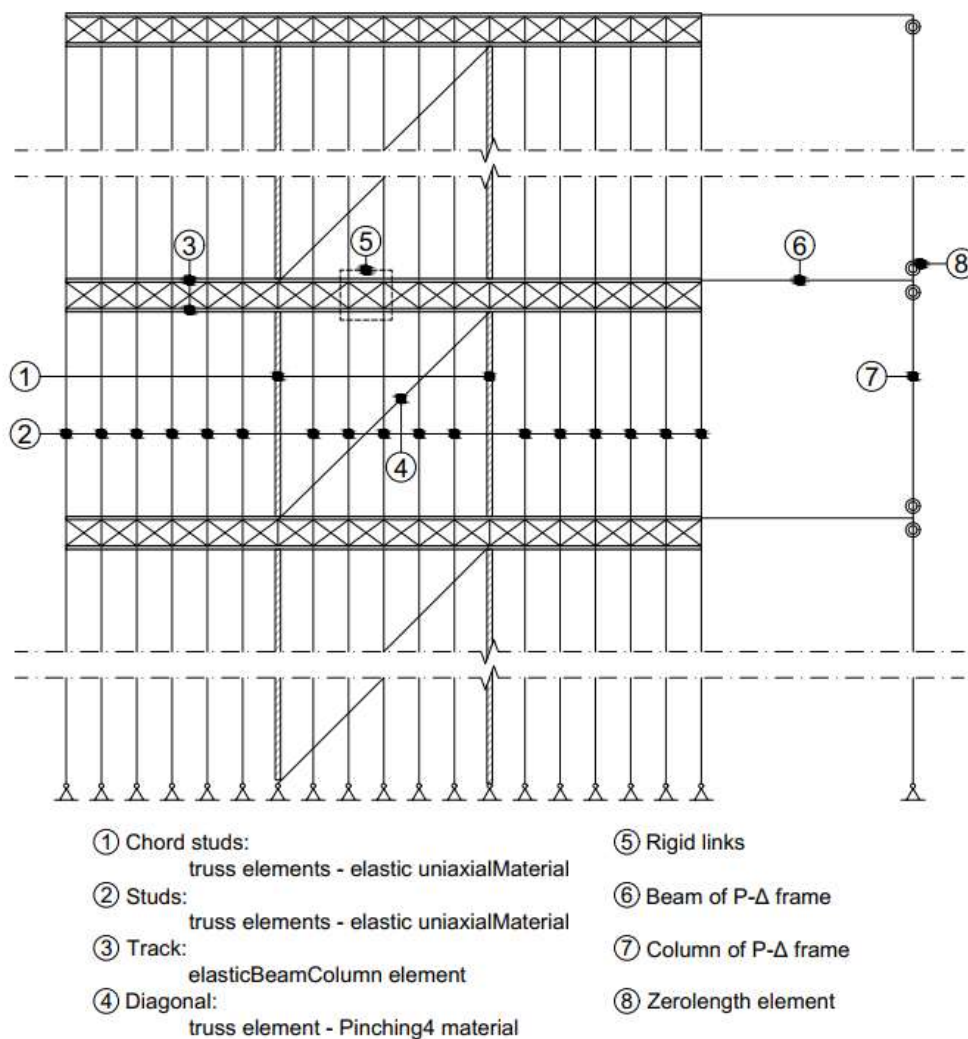
387

388 3.4 Multi-storey frame system

389 The previously verified non-linear OpenSees models of the single strap-braced wall panels
 390 were employed to develop the multi-storey frame models, as depicted in Fig. 9. The chord studs
 391 and the intermediate studs were modelled as elastic truss elements, not transferring any
 392 moments. Assuming that the flooring system provided a rigid diaphragm action, the top and
 393 bottom tracks were modelled as elastic beam-column elements of very high stiffness, connected
 394 at their endpoints with “bar”-type rigid links. At each storey level, the hysteretic behaviour for
 395 the walls was characterised by “Pinching4” material properties assigned to each diagonal
 396 element through the backbone curve parameters obtained from the corresponding ABAQUS
 397 monotonic results (see Section 3.3). In order to account for second-order effects, a P-Δ frame
 398 was attached to the main CFS frame [23,45]. It comprised a leaning column on each level [46],

399 modelled as an elastic beam-column element with a high axial and low flexural stiffness. The
 400 leaning columns were connected to the main frame through rigid beams, modelled by rigid
 401 truss elements, using zero-length hinge elements at both ends (see Fig. 9). The horizontal
 402 movement of the leaning column end nodes was constrained relative to their adjacent nodes of
 403 the main frame at each level.

404 The vertical loads and the storey masses, calculated in Section 2.2, were applied as
 405 uniformly distributed on the track elements at the top of each floor level, while a concentrated
 406 vertical force equal to the total gravity load of each storey was applied at the joints of the P- Δ
 407 frame. Following gravity analysis, a modal analysis was performed. For the dynamic analyses,
 408 a Rayleigh damping value of 2% was considered as suggested by [23,45].



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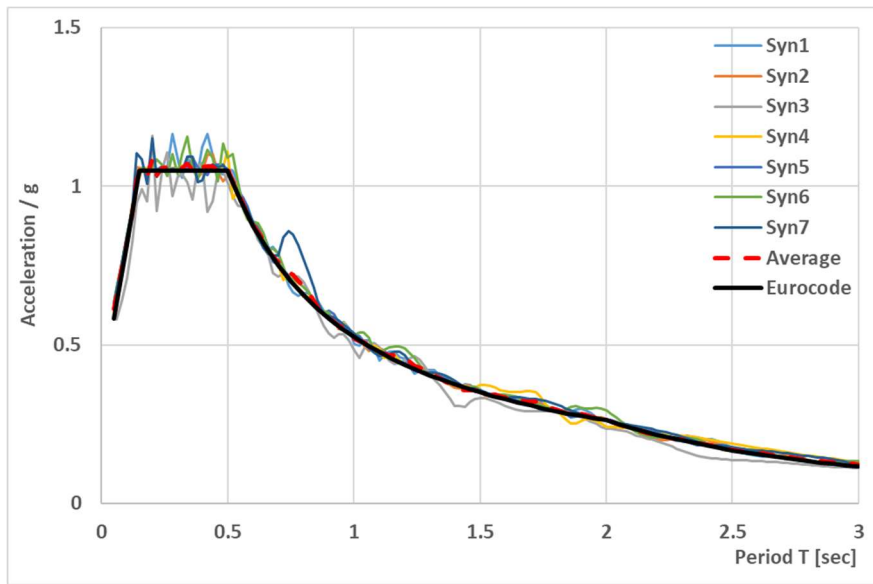
Fig. 9 The strap-braced wall frame model in OpenSees

411 **4 Non-linear time-history analysis**

412 To evaluate the performance of CFS strap-braced wall frame systems and assess the
413 efficiency of the proposed design methodology under seismic excitations, the Eurocode and
414 the proposed design solutions were subjected to non-linear time-history analyses by selecting
415 appropriate artificial and real ground motion records. The ductility demand and capacity of the
416 systems were compared, and their expected overall structural damage, expressed by a global
417 damage index, was calculated.

418 *4.1 Artificial spectrum-compatible ground motion records*

419 The elastic response spectrum per Eurocode 8 [29] was produced for a ground acceleration
420 value of 0.35 g, soil category B, and importance class II. This response spectrum was used for
421 the seismic design of the frames, as discussed in Section 2.2. Seven compatible artificial
422 records (Syn1-Syn7) were then generated with SeismoArtif software [47] to match the design
423 spectrum. In this study, artificial records were initially utilised to assess the seismic
424 performance of the design solutions as representatives of the selected design spectrum [48–50].
425 Fig. 10 demonstrates the good agreement between the response spectra of the artificial records
426 and the Eurocode 8 design response spectrum.



427

428 **Fig. 10** Comparison between elastic response spectra of artificial records and the selected Eurocode 8
 429 design spectrum
 430

430

431 *4.2 Performance Assessment Parameters*

432 For performance-based design and assessment of structures, ASCE/SEI 41-17 [51]
 433 specifies three seismic performance objectives: Immediate Occupancy (IO), Life Safety (LS)
 434 and Collapse Prevention (CP). Each performance objective is represented based on acceptance
 435 criteria corresponding to a specific ground motion intensity level. In this study, the Design-
 436 Basis Earthquake (DBE), corresponding to (LS) performance target, had a $PGA = 0.35\text{ g}$
 437 consistent with the adopted design response spectrum. The Maximum Considered Earthquake
 438 (MCE), associated with (CP) level, had a $PGA = 0.5\text{ g}$ (1.5 times the DBE level as
 439 recommended by ASCE 7-05 [52]). The PGA of the Frequent Earthquake (FE), corresponding
 440 to (IO) performance level, was set to 0.1 g . It should be noted that for practical applications,
 441 these earthquake intensity values should be obtained based on the results of the seismic hazard
 442 assessment of the selected site.

443 In general, the seismic performance of structures is measured based on Engineering
 444 Demand Parameters (EDP), such as inter-storey drift, roof displacement, plastic hinge rotation,

445 and peak storey ductility [53,54]. ASCE/SEI 41-17 [51] prescribes limits for the storey ductility
 446 demand ($\mu_i = \Delta_{\text{inelastic},i} / \Delta_{y,i}$) corresponding to (IO), (LS) and (CP) performance levels.
 447 ($\Delta_{\text{inelastic},i}$) and ($\Delta_{y,i}$) stand for the drift demand and wall drift at yield at storey (i). In this study,
 448 ($\Delta_{y,i}$) was calculated based on the idealised bilinear load-displacement curves, as prescribed in
 449 ASCE/SEI 41-17 [51] , using the results of ABAQUS models. The ductility limits for wall
 450 systems employing dogbone-shaped diagonal straps were set at 3.2, 7.3 and 10.1, for
 451 performance levels (IO), (LS) and (CP), respectively [51].

452 The ductility capacity of the walls at each storey level were calculated in accordance with
 453 the dominant failure mode (i.e. failure of the straps reaching their ultimate strain, or buckling
 454 of the compressive chord stud). To calculate the ductility capacity ($\mu_{c,i} = \Delta_{u,i} / \Delta_{y,i}$), the ultimate
 455 wall drift ($\Delta_{u,i}$) was considered as the drift corresponding either to the respective failure mode
 456 or a 20% drop in the maximum lateral load, whichever occurs first.

457 The efficiency and reliability of the design solutions were also assessed based on the
 458 cumulative global damage model proposed by Krawinkler [55] and Powell and Allahabadi [56],
 459 measuring the change in the dissipated energy relative to the displacement demands. In this
 460 work, the damage index (DI_i) of each storey (i) was selected to be a function of the inter-storey
 461 drift demands [48,49]:

$$DI_i = \sum_{j=1}^N \left(\frac{\Delta\delta_{pj}}{\Delta_{y,i}} \right)^c \quad (5)$$

462 where ($\Delta\delta_{pj}$) is the inelastic inter-storey drift at the j-th excursion, (N) the total number of the
 463 inelastic excursions, ($\Delta_{y,i}$) the inter-storey drift at yield, and (c) is a constant parameter
 464 accounting for the stability of the hysteretic behaviour taken equal 1.5 [48].

465 To quantify the overall damage at the frame-level, the global damage index (DI_g) [48,49]
 466 was used as a weighted average of the damage indices (DI_i) at each storey:

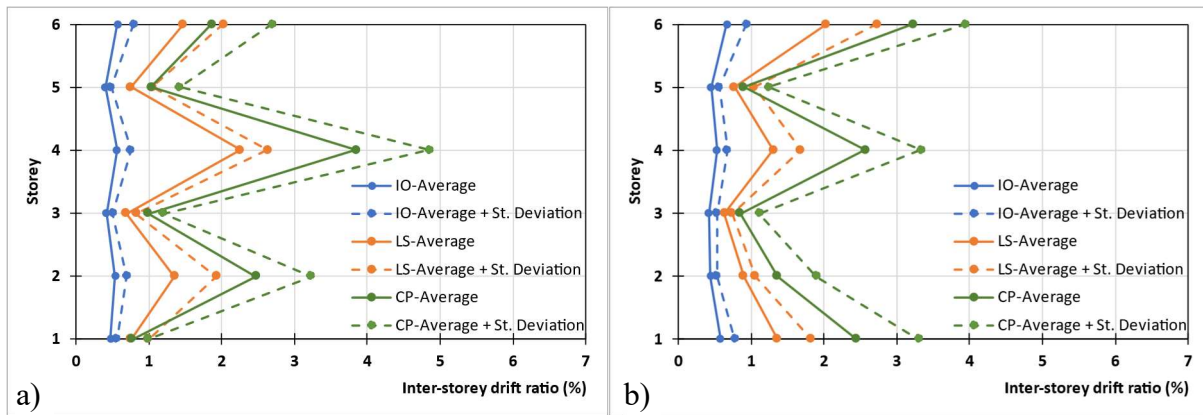
$$DI_g = \frac{\sum_{i=1}^n DI_i \cdot W_{pi}}{\sum_{i=1}^n W_{pi}} \quad (6)$$

467 where (DI_i) is the damage index of each storey (i), (W_{pi}) is the dissipated energy of each storey
468 (i), and (n) is the number of storeys. The global damage index estimates the overall condition
469 of the structure after an earthquake event and takes values between 0 (no damage) and 1
470 (complete damage or failure).

471 **5 Performance evaluation of CFS strap-braced wall multi-storey frames**

472 *5.1 Seismic Performance Assessment*

473 The seismic performance of the design solutions was assessed in terms of the maximum
474 inter-storey drifts and storey ductility demands by scaling the seven artificial records (Syn1 to
475 Syn7) to 0.1 g, 0.35 g and 0.5 g, corresponding to (IO), (LS) and (CP) performance levels (see
476 Section 4.2). As per Eurocode 8 [29], since the results of at least seven time-history analyses
477 are obtained, their average response can be used for design purposes. Therefore, for each of the
478 three performance levels, the inter-storey drift distributions for both design solutions were
479 obtained under the seven spectrum compatible artificial records. Fig. 11 illustrates the average
480 and average plus standard deviation of the results. In general, the results indicate that the inter-
481 storey drift distributions were more uniform in the proposed design solution compared to its
482 code-compliant counterpart, especially under higher earthquake intensity levels. The proposed
483 design method could also reduce the maximum inter-storey drifts, on average, by around 10%
484 and 20% under (LS) and (CP) earthquake intensity levels, respectively.
485



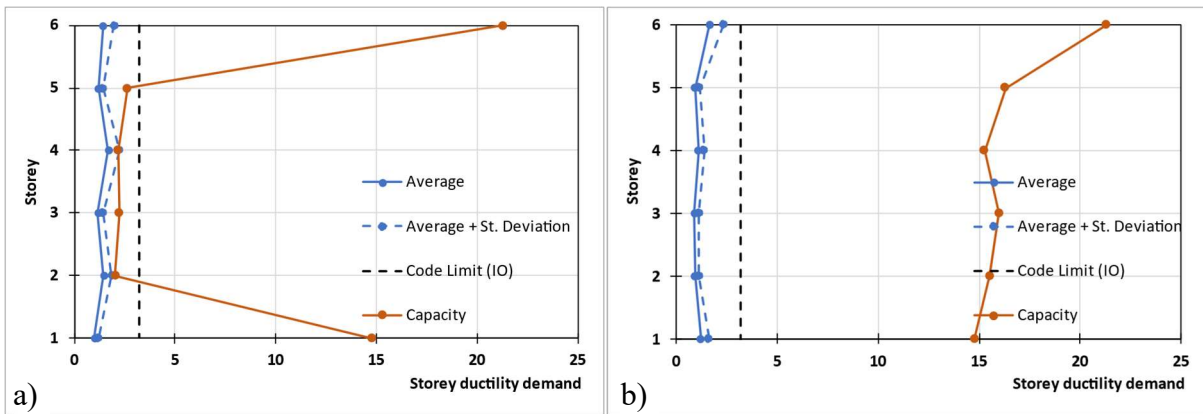
486

487 **Fig. 11** Inter-storey drift distribution of a) Eurocode design and b) Proposed design under seven
 488 artificial spectrum compatible records

489

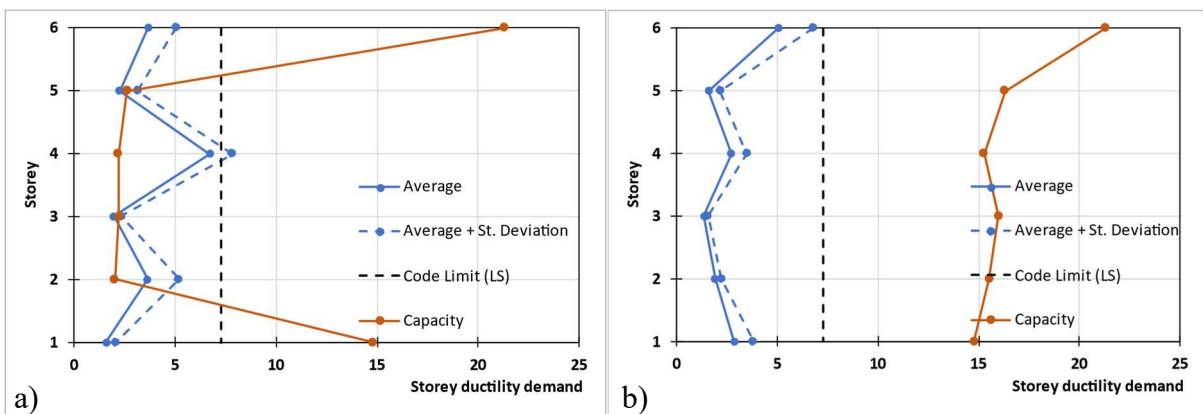
490 Fig. 12 to Fig. 14 display the average and the average plus standard deviation distribution
 491 of the storey ductility demands of the Eurocode and the proposed design solutions for (IO),
 492 (LS) and (CP) performance levels, respectively. The code ductility limits and the ductility
 493 capacity of each storey are also plotted for comparison purposes (see Section 4.2). It is shown
 494 in Fig. 12 that both frames could satisfy the code ductility limit of 3.2 for the (IO) performance
 495 level. The storey ductility capacity values (obtained from ABAQUS models) were also above
 496 the demand values for both cases. However, the proposed design solution provided a
 497 considerably higher safety margin in this case. The results presented in Fig. 13 indicate that,
 498 on average, both frames satisfied the code ductility limit of 7.3 for the (LS) performance level.
 499 However, the average plus standard deviation ductility marginally exceeded this limit at the 4th
 500 storey of the Eurocode design solution. Moreover, it is shown that the average storey ductility
 501 demands at the 2nd and 4th storey of the Eurocode design were considerably higher than their
 502 corresponding capacity values. This indicates that the Eurocode frame exhibited a soft-storey
 503 mode of failure at these storey levels due to the premature buckling of the compressive chord
 504 studs, not allowing the diagonal straps to reach their ultimate capacity. This confirms that
 505 satisfying the code suggested ductility demands by ignoring the effects of vertical loads may
 506 lead to unsafe design solutions. It can be seen that this issue was addressed by using the
 507 proposed design method, as the storey ductility capacities were always considerably higher

508 than the demand values. For the (CP) performance level, Fig. 14 shows that the code ductility
 509 limit of 10.1 was exceeded, by 13% at the 4th storey of the code design solution, while it was
 510 fully satisfied in the proposed design solution. The storey ductility demands at the 2nd, 3rd, 4th
 511 and 5th storey levels of the Eurocode design frame were also exceeded their capacity values,
 512 indicating a premature failure mechanism under this earthquake intensity level. Similar to the
 513 previous case, by adopting the proposed design method, the capacity values were always
 514 considerably above the storey ductility demands at all storey levels.
 515



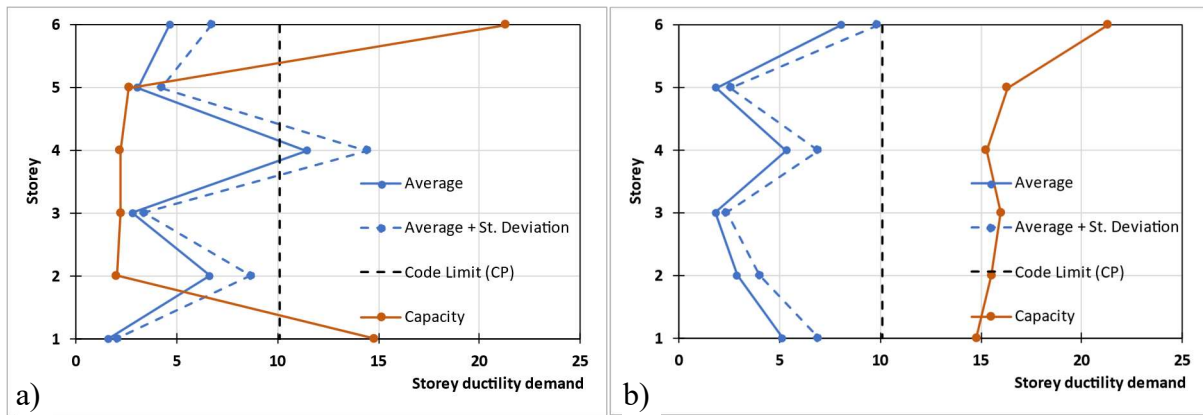
516 a) b)
 517 **Fig. 12** Storey ductility demand and capacity distribution of a) Eurocode design and b) proposed
 518 design under seven artificial spectrum compatible records, IO performance limit

519
 520



521 a) b)
 522 **Fig. 13** Storey ductility demand and capacity distribution of a) Eurocode design and b) proposed
 523 design under seven artificial spectrum compatible records, LS performance limit

524



525

526 **Fig. 14** Storey ductility demand and capacity distribution of a) Eurocode design and b) proposed
 527 design under seven artificial spectrum compatible records, CP performance limit

528

529 *5.2 Non-linear Incremental Dynamic Analysis (IDA)*

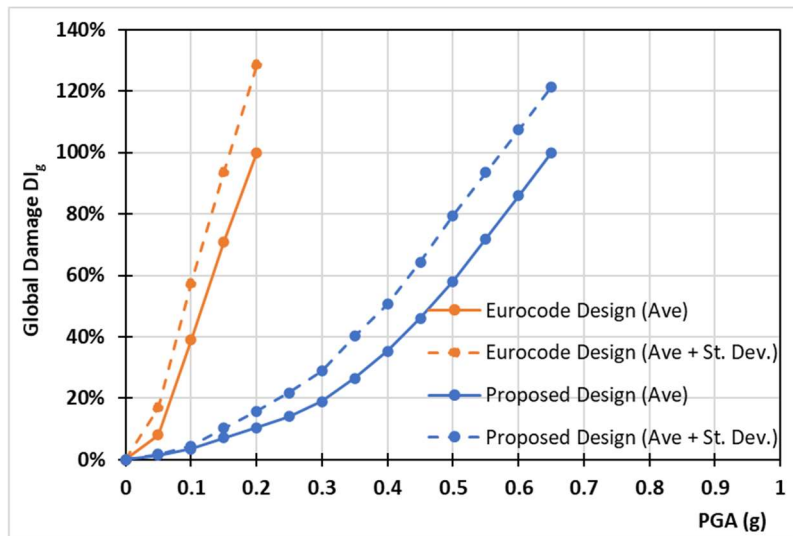
530 Incremental Dynamic Analysis (IDA) is a numerical method to assess the performance
 531 and vulnerability of structures under seismic loads [54,57]. The studied structure is subjected
 532 to ground motion records scaled to a progressively increased intensity, and its structural
 533 performance is evaluated using a response parameter such as maximum roof displacement or
 534 global damage index. In this study, IDA was performed on the Eurocode and the proposed
 535 design solutions subjected to the seven artificial spectrum-compatible ground motion records,
 536 as was described in Section 4.1. The PGA of the input earthquakes was considered as the
 537 intensity parameter to be consistent with the limits used for the ASCE/SEI 41-17 [51]
 538 performance targets (see Section 4.2).

539 The average and average plus standard deviation of the global damage index (DI_g) values
 540 for the Eurocode and the proposed design solutions subjected to the artificial spectrum-
 541 compatible records at different intensity levels are shown in Fig. 15. Due to the low ductility
 542 capacity of some storey levels, the Eurocode design frame exhibited a high level of damage
 543 even under low intensity records. It is shown that the global damage index for the Eurocode
 544 design solution was around 40% at $PGA = 0.10$ g (IO performance level), while the frame was
 545 completely damaged under the PGA levels above 0.20 g. This clearly indicates that the frame

546 did not comply with the performance targets of the seismic design codes. As discussed before,
 547 this is mainly attributed to the buckling of the compressive studs due to the negligence of the
 548 P- Δ secondary moment effects during the code-base design process.

549 The improved proposed design frame, on the other hand, experienced considerably lower
 550 damage compared to its code-compliant counterpart at all earthquake intensity levels. By
 551 adopting the proposed design method, the frame exhibited a global damage index of 4%, 26%
 552 and 58% at PGA levels of 0.10 g (IO performance level), 0.35 g (LS performance level) and
 553 0.5 g (CP performance level), respectively. The full damage ($DI_g=100\%$) in this case was
 554 reached only at PGA levels above 0.65 g. In general, these results confirm that the proposed
 555 design solution fulfils the expected performance targets for typical building structures.

556
 557



558

559 **Fig. 15** Global damage index (DI_g) for the Eurocode and proposed design solution frames subjected to
 560 artificial spectrum-compatible ground motion records

561 *5.3 Seismic performance under real ground motion records*

562 In this section, the seismic performance of the Eurocode and the proposed design solutions
 563 are assessed under compatible real ground motion records. A set of twenty earthquakes from
 564 the Pacific Earthquake Engineering Research Centre (PEER) ground motion database [58]

565 were selected to be compatible with the selected elastic design response spectrum of
566 Eurocode 8 [29]. All the accelerograms were obtained for a combination of near and far-field
567 earthquakes with magnitudes between 6.8 and 7.6 (i.e. medium to strong earthquake records).
568 The characteristics of the selected records are summarised in Table 5, and their respective
569 response spectra are compared with the Eurocode design spectrum in Fig. 16. In Table 5, (R_{rup})
570 is the site distance from the epicentre, and (V_{s30}) is the average shear wave velocity, with the
571 search range set between 360-800 m/sec, to match ground type B, per Eurocode 8 [29]. It is
572 shown in Fig. 16 that the average of the earthquake response spectra of the selected records
573 compares very well with the Eurocode design spectrum used in the design process of the frames
574 (see Section 2.1). Therefore, these records were used without scaling.

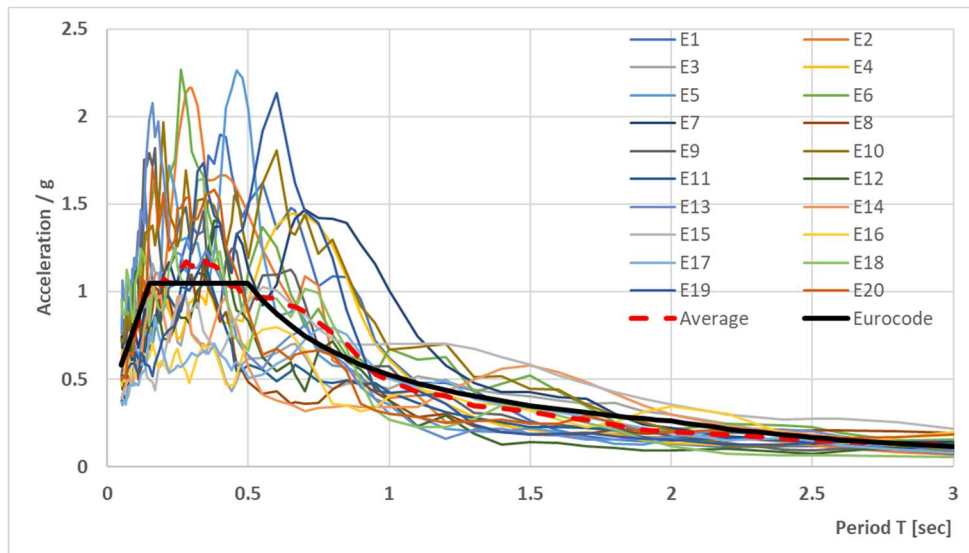
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Table 5 Real ground motion records

Record name	Earthquake Name	Year	Magnitude	R_{rup} (km)	V_{s30} (m/sec)
E1	"Loma Prieta"	1989	6.93	10.72	476.54
E2	"Loma Prieta"	1989	6.93	3.85	462.24
E3	"Loma Prieta"	1989	6.93	8.5	380.89
E4	"Cape Mendocino"	1992	7.01	8.18	422.17
E5	"Kobe_ Japan"	1995	6.9	7.08	609
E6	"Chi-Chi_ Taiwan"	1999	7.62	5.8	624.85
E7	"Chi-Chi_ Taiwan"	1999	7.62	7.08	468.14
E8	"Chi-Chi_ Taiwan"	1999	7.62	2.74	614.98
E9	"Chi-Chi_ Taiwan"	1999	7.62	8.2	443.04
E10	"Chi-Chi_ Taiwan"	1999	7.62	10.97	363.99
E11	"Chi-Chi_ Taiwan"	1999	7.62	9	671.52
E12	"Chi-Chi_ Taiwan"	1999	7.62	45.18	446.63
E13	"Manjil_ Iran"	1990	7.37	12.55	723.95
E14	"Cape Mendocino"	1992	7.01	18.31	459.04
E15	"Cape Mendocino"	1992	7.01	19.32	387.95
E16	"Chuetsu-oki_ Japan"	2007	6.8	11.94	383.43
E17	"Chuetsu-oki_ Japan"	2007	6.8	16.86	561.59
E18	"Chuetsu-oki_ Japan"	2007	6.8	20.03	561.59
E19	"Chuetsu-oki_ Japan"	2007	6.8	20	561.59
E20	"Iwate_ Japan"	2008	6.9	12.85	512.26

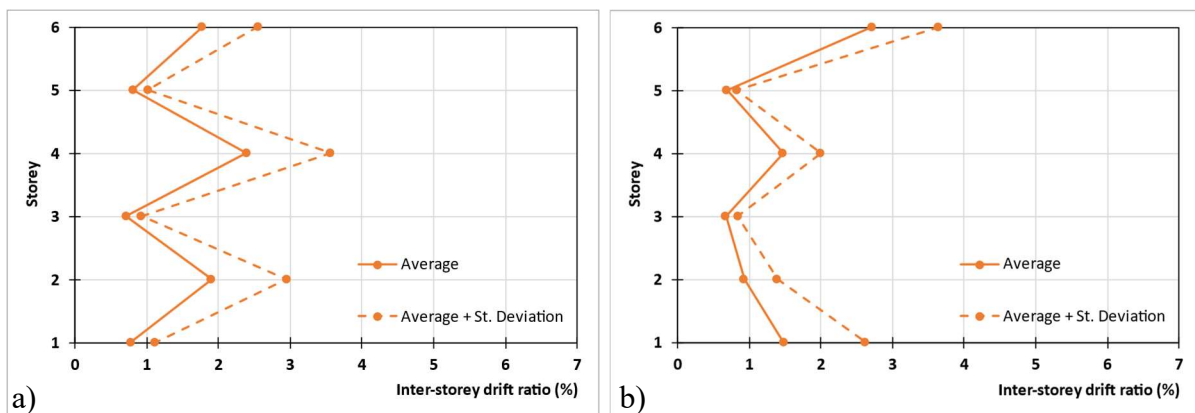
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578

579 **Fig. 16** Comparison between the response spectra of the real earthquakes and the elastic design
 580 response spectrum of Eurocode

581 The Eurocode and the proposed design frames were subjected to the set of the real
 582 earthquake records, and the average and the average plus standard deviation of inter-storey
 583 drift distributions were calculated as displayed in Fig. 17. It can be noted that the average inter-
 584 storey drift patterns were almost similar to those under artificial spectrum-compatible ground
 585 motions, which confirms the general agreement between the artificial and real records. The
 586 Eurocode design frame reached a maximum average inter-storey drift value of 2.4% (average
 587 plus standard variation of 3.6 %), while the maximum average inter-storey drift value slightly
 588 increased to 2.7% (average plus standard variation of 3.6 %) in the proposed design solution.
 589



590

591 **Fig. 17** Inter-storey average drift distribution for (LS) performance limit of a) the Eurocode design
 592 and b) the proposed design solutions subjected to real ground motion records

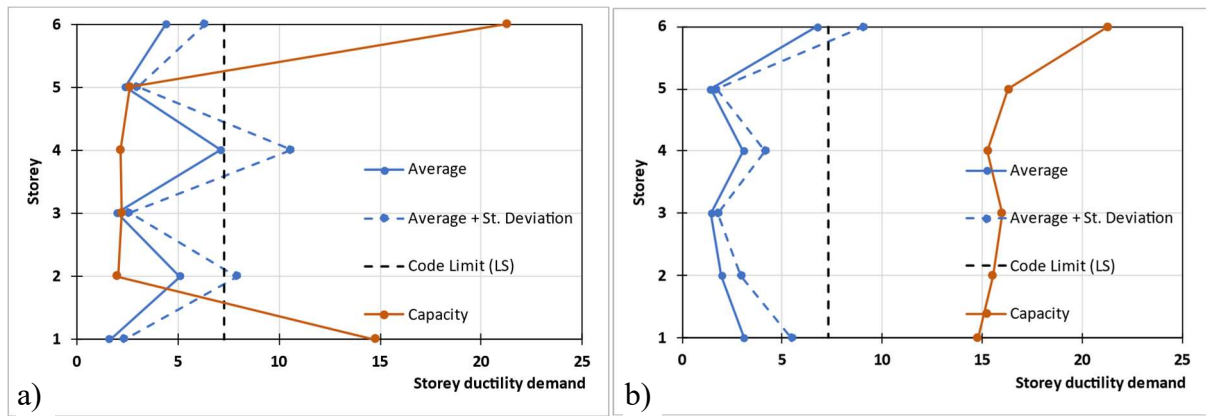
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The average and the average plus standard deviation of storey ductility demands for the Eurocode and the proposed design solutions are shown in Fig. 18. The average storey ductility demands for the Eurocode design frame were always below the code limit of 7.3 for the (LS) performance level. However, at the 2nd and 4th storey levels, they exceeded the ductility capacity by 2.5 and 3.2 times, reaching values of 5.1 and 7.2, respectively. As discussed before, this can lead to extensive damage in the Eurocode design structure under strong earthquake events.

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The average storey ductility demand for the proposed design solution reached a maximum value of 6.8, which means the frame satisfied the code (LS) performance level. Besides, the results indicate that in the proposed design frame, the average storey ductility demands were always well below the storey ductility capacity values. It is shown that the adopted design methodology could significantly increase the ductility capacity of the storeys that initially suffered from the premature failure of the compressive chord studs. These results are consistent with those observed under artificial records in Section 5.1.

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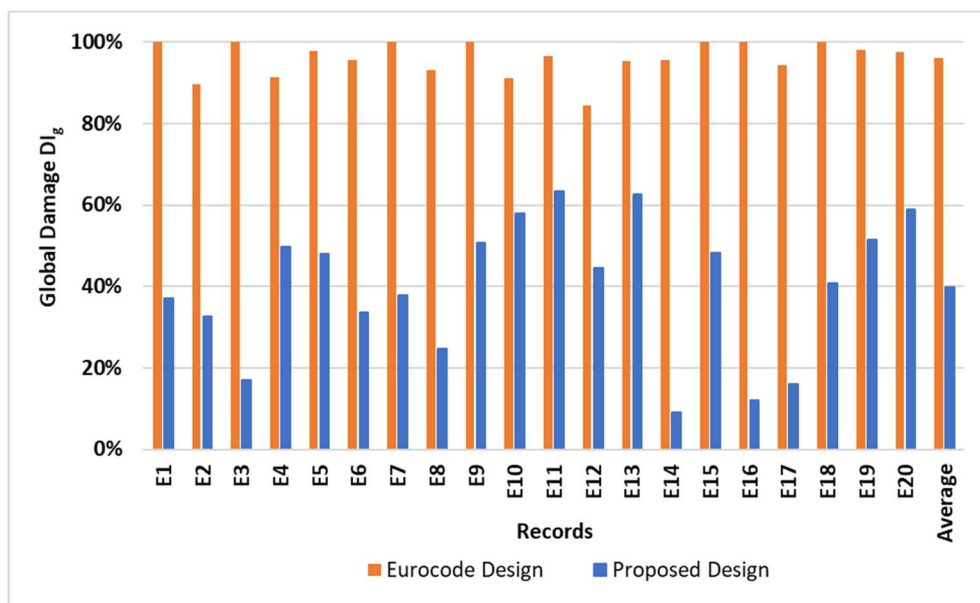


610 **Fig. 18** Average storey ductility demand and capacity for (LS) performance limit of a) the Eurocode
611 design and b) the proposed design solutions subjected to real ground motion records

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The global damage indices (DI_g) for the Eurocode and the proposed design solutions were also calculated under the set of 20 real ground motion records (E1 to E20), as plotted in Fig. 19. For the Eurocode design frame, the global damage index was very high for all the selected

616 records, indicating an unacceptable level of damage leading to complete collapse in most cases.
 617 The global damage index for the proposed design solution ranged between 10% and 63%, with
 618 an average value of 40%. This significant improvement in the seismic performance of the
 619 modified frame is attributed to the higher ductility capacity of the system achieved by taking
 620 into account the effects of vertical loads on the lateral load capacity and ductility of the wall
 621 panels in the design process. The results of this study, in general, highlight the efficiency of
 622 the proposed method, which should prove useful in the preliminary design of multi-storey CFS
 623 strap-braced frames in seismic regions.
 624



625
 626 **Fig. 19** Global damage index (DI_g) for the Eurocode and proposed design solution frames subjected to
 627 a set of real ground motion records
 628

629 **6 Summary and Conclusions**

630 This study aimed to investigate the efficiency of the Eurocode and a newly developed
 631 methodology, which accounts for the secondary moments due to P- Δ effects, amplified by the
 632 presence of additional vertical loading, for the seismic design of multi-storey CFS strap-braced
 633 stud wall frames. Detailed non-linear FE models of single strap-braced wall panels were
 634 developed in ABAQUS and validated against available experimental data. The validated

635 models were then adopted to develop equivalent hysteretic models in OpenSees for different
636 cross-section sizes and vertical load levels. Subsequently, a case study 6-storey CFS frame was
637 designed per Eurocode 8 and the proposed methodology. The seismic performance of the
638 frames was assessed in terms of the maximum inter-storey drifts and storey ductility demands
639 under a set of artificial spectrum-compatible records scaled to (IO), (LS) and (CP) performance
640 earthquake intensity levels. For further comparison, an incremental dynamic analysis (IDA)
641 was also conducted by using a global cumulative damage index to assess the overall
642 performance of the frames. Finally, the efficiency of the two design methods was demonstrated
643 under a set of 20 real spectrum-compatible records. The study led to the following main
644 conclusions:

- 645 • The Eurocode 8 design solution could satisfy the ASCE/SEI 41-17 ductility demand
646 limits for (IO) and (LS) performance levels, while the ductility limit for (CP)
647 performance level was exceeded by 13%. The proposed design methodology yielded a
648 more uniform inter-storey drift distribution compared to its Eurocode 8 counterpart,
649 especially for the higher intensity levels, and met all the ductility requirements with a
650 higher safety margin.
- 651 • It was shown that ignoring the effects of vertical loads may lead to unsafe design
652 solutions. The Eurocode frame was completely damaged under the PGA levels above
653 0.20 g, due to the premature buckling of the compressive chord studs leading to
654 unacceptable ductility capacities at some storeys. The proposed design experienced
655 significantly lower damage at all earthquake intensity levels and exhibited a global
656 damage index of 4%, 26% and 58% at PGA levels corresponding to IO, LS and CP
657 performance levels, respectively.
- 658 • Both frames could satisfy the ASCE/SEI 41-17 (LS) ductility demand limits under a set
659 of design compatible real ground motion records. While the Eurocode design frame

660 completely collapsed in most cases, the global damage index for the proposed design
661 solution ranged between 10% and 63%, with an average value of 40%.

- 662 • These results highlight the efficiency of the proposed methodology for the preliminary
663 design of multi-storey CFS strap-braced frames in seismic regions.
- 664 • However, further studies are needed to investigate the efficiency of the methodology
665 for other strap-braced wall configurations and topologies.

666

667 **Declaration of competing interest**

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

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