

Hysteretic Behavior of Moment-Resisting Frames Considering Slab Restraint and Framing Action

Hammad El Jisr¹, S.M. ASCE; Ahmed Elkady²; and Dimitrios G. Lignos^{3*}, M. ASCE

Abstract: This paper examines the influence of the framing action and slab continuity on the hysteretic behavior of composite-steel moment-resisting frames (MRFs) by means of high-fidelity continuum finite element (CFE) analyses of two-bay sub-systems and typical cruciform subassemblies. The CFE model, which is made publicly available, is thoroughly validated with available full-scale experiments and considers variations in the beam depth and the imposed loading history. The simulation results suggest that beams in sub-systems may experience up to 25% less flexural strength degradation than those in typical subassemblies. This is due to local buckling straightening from the slab continuity and framing action evident in sub-systems. For the same reason, beam axial shortening due to local buckling progression is up to five times lower in sub-systems than in subassemblies, which is consistent with field observations. While the hysteretic behavior of interior panel zone joints is symmetric, exterior joint panel zones in sub-systems experience large asymmetric shear distortions regardless of the employed lateral loading history. From a design standpoint, it is found that the probable maximum moment in deep and slender beams ($d_b \geq 700\text{mm}$) may be up to 25% higher than that predicted by current design provisions with direct implications to capacity design of steel MRFs. The 25% reduction in the shear stud capacity as proposed by current seismic provisions is not imperative for MRFs comprising intermediate to shallow beams and/or featuring a high degree of composite action ($\eta > 80\%$) as long as ductile shear connectors are employed.

Keywords: Composite-steel moment resisting frames; Slab restraint; Frame continuity; Continuum finite element analysis; Cyclic deterioration; Panel zone shear distortion; Lateral load protocol; Shear stud behavior

¹ PhD candidate, RESSLab, ENAC, École Polytechnique Fédérale de Lausanne (EPFL), Switzerland. E-mail: hammad.eljisr@epfl.ch

² Lecturer, University of Southampton, Southampton, United Kingdom. E-mail: a.elkady@soton.ac.uk (formerly, Post-doctoral research scientist, RESSLab, ENAC, École Polytechnique Fédérale de Lausanne (EPFL), Switzerland)

^{3*} (Corresponding Author) Associate professor, RESSLab, ENAC, École Polytechnique Fédérale de Lausanne (EPFL), Switzerland. E-mail: dimitrios.lignos@epfl.ch

27 **Introduction**

28 The 1994 Northridge and 1995 Kobe earthquakes led to a paradigm shift in the seismic design
29 of steel moment resisting frames (MRFs). As part of the SAC project (Mahin 1998), multiple
30 testing programs were conducted on beam-to-column subassemblies (Engelhardt et al. 2000;
31 FEMA 2000b; Ricles et al. 2002; Sumner and Murray 2002; Tremblay et al. 1997; Uang et al.
32 2000; Zhang et al. 2004). These tests formed the basis for the development of today's pre-qualified
33 beam-to-column connections for seismic applications in the US (AISC 2016a). A concerted effort
34 is currently underway in Europe (Landolfo et al. 2018) regarding the same matter.

35 The significant majority of the tests conducted as part of the SAC project involved
36 subassemblies with T- or cruciform-shaped configurations. While these subassemblies may be
37 convenient for physical testing due to their overly-simplified boundary conditions, they do not
38 represent reality at damage states associated with large inelastic deformations (Cordova and
39 Deierlein 2005; Zerbe and Durrani 1989). In particular, beams in cruciform subassemblies are free
40 to shorten axially (Civjan et al. 2001; MacRae et al. 2013) after the formation of local buckling
41 within the anticipated dissipative zone of a steel beam. This is not evident in system-level tests
42 (Cordova and Deierlein 2005; Del Carpio et al. 2014) and field observations (Clifton et al. 2011;
43 Okazaki et al. 2013), where beam local buckling is delayed due to the axial restraint provided by
44 the slab continuity (Cordova and Deierlein 2005; Donahue et al. 2017; FEMA 2000a; Herrera et
45 al. 2008). Moreover, the floor slab and adjacent columns in buildings provide restraint to the beam
46 and inhibit axial shortening (PEER/ATC 2010). This may result in an appreciable increase in the
47 plastic rotation capacity of the steel beam (FEMA 2000a; Kwasniewski et al. 2002). Although
48 inconclusive, El Jisr et al. (2019) highlighted that the plastic rotation capacity of composite steel
49 beams directly deduced from system-level or sub-system tests may be at least two times larger
50 than that deduced from beams in cruciform configurations.

51 The effect of the slab axial restraint may have significant implications on nonlinear modeling
52 of steel MRF beams. Common numerical modeling approaches include point plastic hinge models
53 as well as distributed finite-element approaches (Deierlein et al. 2010). While point hinge (Elkady
54 and Lignos 2014; Ibarra et al. 2005; Lignos and Krawinkler 2011; Rassati et al. 2004) and resultant
55 section models (El-Tawil and Deierlein 2001; Mehanny and Deierlein 2000) for composite steel
56 beams are available in the literature, they have been established on the basis of subassembly tests.

57 Hence, potential differences, due to the framing action, in the hysteretic behavior of beams
58 between interior and exterior joints of a steel MRF are ignored. Furthermore, these models
59 typically neglect the interface slip between the steel beam and the concrete slab. These effects may
60 be captured with fiber models (Amadio and Fragiaco 1993; Ayoub 2005; Ayoub and Filippou
61 2000; Bursi et al. 2005; Bursi and Ballerini 1996; Gattesco 1999; Salari and Spacone 2001). Albeit
62 these models are computationally efficient, they require effective stress-strain formulations with
63 softening to trace strength and stiffness deterioration (Kolwankar et al. 2018; Suzuki and Lignos
64 2018). In the absence of comprehensive experimental data, continuum finite element (CFE)
65 models offer a rational alternative to quantify the aforementioned effects. Past studies involving
66 CFE models (Alashker et al. 2010; Elkady and Lignos 2015b, 2018a; Zhang et al. 2004; Zhou et
67 al. 2007) focused mostly on the dependence of strength and stiffness deterioration of steel members
68 on nonlinear geometric instabilities (e.g., local and/or lateral torsional buckling). To the best of the
69 authors' knowledge, there are no comprehensive studies to elucidate the physical mechanisms
70 associated with the slab axial restraint at interior and exterior beam-to-column joints within a sub-
71 system (entire story) and/or structural system.

72 From a seismic design standpoint, headed shear studs transfer seismic inertia forces through the
73 slab into the MRF steel beams. Early degradation in the shear stud strength results in the loss of
74 this load-transfer mechanism, thereby triggering loss of composite action (Cheng and Chen 2005;
75 Civjan et al. 2001; Leon et al. 1998). As a precaution against severe shear strength degradation of
76 the studs, current seismic provisions (AISC 2016b; CEN 2004a) propose a 25% reduction in the
77 stud design shear resistance. Albeit this reduction may be rational in steel MRFs comprising deep
78 beams (depths larger than 400mm), it is not justifiable in prospective steel MRF designs
79 comprising shallow composite beams with a high degree of composite action, $\eta \geq 80\%$ (η is the
80 ratio of the actual number of shear studs to that required to achieve full composite action). This
81 perception has been mostly put in place based on cyclic push-out tests (Bursi and Gramola 1999;
82 Civjan and Singh 2003; Zandonini and Bursi 2002). Although informative, these tests do not
83 replicate the actual stress state and boundary conditions in the slab due to bending, nor do they
84 account for the force redistribution between the studs (Schafer et al. 2019; Sjaarda et al. 2018;
85 Suzuki and Kimura 2019).

86 This paper addresses all the aforementioned issues by means of CFE analyses. The proposed
87 CFE modeling approach, which is validated on the basis of composite subassembly tests, explicitly

88 accounts for the synergy between the composite slab and the steel beams. The modeling approach
89 is extended to two-bay sub-systems to comprehend the influence of the slab axial restraint and
90 framing action on the hysteretic behavior of beams and panel zones in interior/exterior beam-to-
91 column joints. These sub-systems comprise beams with depths representative of both the North
92 American and European seismic design practice. Aspects associated with the shear stud resistance
93 in contemporary designs of composite steel MRFs are discussed.

94 **Proposed Continuum Finite Element Modeling Approach**

95 This section discusses the CFE modeling specifics of a typical beam-to-column subassembly
96 with a composite floor slab as illustrated in Fig. 1(a). The modeling approach is validated with
97 data from a full-scale subassembly test (Zhang et al. 2004) featuring fully-restrained beam-to-
98 column connections with reduced beam sections (RBS). The commercial finite element software
99 Abaqus 6.14 (Abaqus 2014) is used for this purpose. Referring to Fig. 1, column and beam regions
100 within contact zones are meshed with first-order brick elements with incompatible modes, *C3D8I*.
101 These elements are suitable for nonlinear analysis involving contact (Selamet and Garlock 2010).
102 They are fully integrated with additional internal degrees of freedom to eliminate shear locking
103 and they capture bending with an accuracy similar to that of quadratic elements. A structured mesh
104 is implemented in the beam's anticipated plastic hinge location to produce elements with
105 reasonable aspect ratios. Three elements per flange thickness are considered as recommended by
106 Bursi and Jaspart (1998) for flexure-dominated problems. The remaining beam and column regions
107 are modeled using four-node double-curved *S4R* shell elements with five integration points along
108 the element thickness based on the recommendations by (Elkady and Lignos 2018a). A shell-to-
109 solid coupling constraint is used to connect the shell edge regions of the beam to the column flange.
110 The concrete slab is modeled using eight-node first-order brick elements with reduced integration,
111 *C3D8R*. Five elements across the slab thickness are considered to provide satisfactory performance
112 against hourglassing (Genikomsou and Polak 2015). Slab rebar and wire mesh reinforcement are
113 modeled using two-node linear truss elements, *T3D2*. The steel deck is modeled using four-node
114 membrane elements, *M3D4R*.

115 Shear studs connecting the slab to the transverse beams are modeled with two-node linear beam
116 elements, *B31*. The cross-sectional area of these elements is modified to make it equivalent to the
117 actual shear stud strength and stiffness (Baskar et al. 2002; Liang et al. 2005). The shear studs

118 between the main beams and the slab may exhibit a pinched hysteretic degrading response. To
 119 capture this response, the interface slip should be modeled using a nonlinear load-slip behavior
 120 (Ayoub and Filippou 2000; Bursi et al. 2005). For this purpose, a user-defined element (VUEL) is
 121 developed by the authors and implemented as shown in Fig. 1(d). This VUEL, which is publicly
 122 available from https://github.com/eljistr/IMK_Pinching_VUEL, employs the modified Ibarra-
 123 Medina-Krawinkler (IMK) deterioration model (Ibarra et al. 2005; Lignos and Krawinkler 2011).
 124 The model assumes a pinched hysteretic behavior that explicitly simulates the effects of stiffness,
 125 strength, post-capping strength and accelerated reloading stiffness deterioration (Ibarra et al. 2005).
 126 It is loading-history independent and assumes a reference inherent hysteretic energy dissipation
 127 capacity regardless of the applied protocol. The set of parameters that define the deterioration
 128 model are calibrated with cyclic push-out tests available in literature. Past conventional cyclic
 129 push-out tests have demonstrated severe degradation in the shear studs (Bursi and Gramola 1999;
 130 Civjan and Singh 2003; Zandonini and Bursi 2002). However, they do not accurately replicate the
 131 mechanical behavior of the shear stud connectors under fully reversed cyclic loading. More
 132 importantly, they do not consider the force redistribution occurring in the studs once they
 133 experience cyclic deterioration. For this reason, the shear studs are calibrated with recently
 134 conducted cyclic-push out tests that account for the stress state in the slab under reversed cyclic
 135 loading (Suzuki and Kimura 2019). These tests were subjected to symmetric loading protocols,
 136 which impose far higher inelastic demands to studs than non-symmetric loading protocols. In that
 137 respect, the calibration is on the conservative side. Figure 2 shows a calibration of the hysteretic
 138 behavior of a cluster of four 19mm shear studs. Based on this calibration, the following parameters
 139 are obtained for a single stud (positive and negative superscripts refer to the stud parameters when
 140 the slab is under compression and tension, respectively): the ultimate shear strengths $Q_u^+ = 82\text{kN}$
 141 and $Q_u^- = 36\text{kN}$, the effective yield strengths $Q_y = 90\%$, the pre-capping slip capacities $s_p^+ =$
 142 6mm and $s_p^- = 10\text{mm}$, the post-capping slip capacities $s_{pc}^+ = 11\text{mm}$ and $s_{pc}^- = 5\text{mm}$, the ultimate
 143 slip capacities $s_u^\pm = 15\text{mm}$, the strength and stiffness deterioration parameters $\lambda_s = 40$ and
 144 $\lambda_k = 15$, the deterioration rate parameters $D^\pm = 1.0$, and the parameters that define the break point
 145 of the pinching model $\kappa_d = 0.4$ and $\kappa_f = 0.2$.

146 The steel material multi-axial constitutive relationship for beams and columns is based on the
 147 well-established Voce-Chaboche multi-axial plasticity model (Lemaitre and Chaboche 1994; Voce
 148 1948). The input model parameters are adopted based on studies by Sousa and Lignos (2018) for

149 A992 Gr. 50 steel (ASTM 2015). An elastic perfectly-plastic material model is assigned to the
150 steel deck and slab reinforcement.

151 The concrete behavior under cyclic loading is simulated through the concrete damaged
152 plasticity (CDP) model, available in Abaqus 6.14 (Abaqus 2014). The yield function of this model
153 under multiaxial stress state accounts for damage in the concrete (Lee and Fenves 1998; Lubliner
154 et al. 1989). The plastic flow potential is defined using the Drucker-Prager hyperbolic function.
155 The parameters recommended by Goto et al. (2010) are implemented in the developed CFE model.
156 The stress-strain relation of concrete under compression is defined according to Carreira and Chu
157 (1985) and Baskar et al. (2002). The concrete compressive strength is $f'_c = 32\text{MPa}$. Under tension,
158 a linear elastic behavior is assumed up to the concrete tensile strength, $f_t = 10\%f'_c$ (Matsumura
159 and Mizuno 2007). A tension stiffening strain of 0.1 is employed (Baskar et al. 2002; Rex and
160 Easterling 2000). Stiffness degradation mechanisms are incorporated in the CDP model through
161 compressive and tensile damage variables (Goto et al. 2010).

162 Referring to Fig. 1(a), the pinned boundary conditions assumed at the main beams and column
163 ends correspond to those expected at the inflection point locations in a typical MRF under lateral
164 loading. Out-of-plane movement and twisting are restrained at the main beams and column ends
165 at the indicated points shown in Fig. 1(a). The transverse floor beams supporting the floor slab are
166 connected to the main beams via a conventional shear tab connection (see Fig. 1(c)). A tie
167 constraint is used to idealize this connection for the translational degrees of freedom. Therefore,
168 the connection can resist moment under strong-axis bending only. This assumption is based on the
169 fact that the shear-tab connection is not an ideal pin and has a non-negligible strong-axis rotational
170 strength and stiffness (Liu and Astaneh-Asl 2004). Furthermore, the strong-axis flexural demand
171 on the connection (due to twisting) is low and is mainly resisted by the diagonal brace and the slab.
172 On the other hand, under weak axis bending, the rotational stiffness and flexural strength of the
173 shear tab connection is negligible. The out-of-plane movement of the main beam's bottom flanges
174 is prevented with diagonal braces connected to the transverse beams. Modeling of these braces is
175 simplified by employing a kinematic coupling constraint as shown Fig. 1(c).

176 The steel beams are rigidly connected to the column through a surface-based tie constraint.
177 Continuity plates are fully tied to the column web and flanges, while doubler plates are tied to the
178 column web at their edges. Plug welds are modeled using connector elements with an influence
179 radius equal to that of the plug weld radius, and fully-constrained degrees of freedom (see Fig.

180 1(b)). The shear studs are connected to the beam and slab through multi-point beam constraints. A
181 perfect bond is assumed between the concrete slab, and the rebar reinforcement. The inner surface
182 of the steel deck is fully tied to the concrete slab.

183 Both the restraint provided by the slab to the top beam flanges, and the bearing of the slab on
184 the column flanges are simulated using a general contact interaction. The interface action between
185 the slab and the steel components consists of a hard contact relationship with balanced master-
186 slave weighing and allowed separation. The friction behavior is expressed using a Coulomb model
187 with a steel-to-concrete friction coefficient, $\mu = 0.2$ (Johansson and Gylltoft 2002).

188 Local and global imperfections are introduced in the dissipative zones (i.e., RBS region) to
189 properly trigger nonlinear geometric instabilities based on the modeling procedures proposed by
190 (Elkady and Lignos 2018a). Residual stresses are also modeled using the distribution proposed
191 by Young (1972).

192 **Validation of the Modeling Approach**

193 The proposed CFE modeling approach is validated with the subassembly specimen, SPEC3
194 from Zhang et al. (2004). The specimen features W36x150 main girders, a W27x194 column,
195 W14x22 transverse beams and a 133mm (5.25") slab. The floor slab is 1219mm (4') wide, with a
196 305mm (12") overhang. It consists of an 83mm (3.25") concrete fill (32 MPa) on top of a 51mm
197 (2") deep Vulcraft 2VLI steel deck, oriented such that the ribs are parallel to the main girders. The
198 slab reinforcement includes a W4xW4 welded wire mesh, as well as No. 3 (9.5mm) and No. 4
199 (12.7mm) bars. Nine 19mm (0.75") shear studs connect each main girder to the slab. A single
200 12.7mm (0.5") doubler plate is welded to the column web. The specimen was subjected to a cyclic
201 symmetric loading history (SAC Joint Venture 1997) at the column tip.

202 The nonlinear quasi-static analysis is run at EPFL's high performance computing center (Fidis
203 Cluster) using a Message Passing Interface-based domain decomposition parallel implementation.
204 The Abaqus/Explicit dynamic analysis procedure is employed. This procedure has a robust contact
205 functionality to solve very complex contact problems (Prior 1994). This is critical for simulating
206 the slab restraint to the top beam flange and the slab bearing on the column. In particular, the
207 loading rate is assumed to be sufficiently small to ensure that the inertial force is nearly zero (i.e.,
208 equivalent to static loading). The main drawback of the explicit solution technique is that the time
209 step is limited by the size of the stable time-increment, $t_{stable} \leq 2/\omega_{max}$, where ω_{max} is the

210 highest element eigenfrequency in the model. To overcome this shortcoming, the stable time-
211 increment is increased through mass-scaling. Quasi-static response is verified through the
212 equilibrium of static forces and the energy balance in the model. Referring to Fig. 3, the ratio of
213 the kinetic and viscous energies to the internal energy is less than 5% and the total energy in the
214 model is nearly zero (Chung et al. 1998; Prior 1994). Artificial strain energy due to hourglassing
215 control as well as distortion control dissipation energy are examined and found to be negligible.

216 Figures 4(a and b) shows a comparison between the measured and simulated hysteretic response
217 of the composite steel beam. In this figure, M_b is the beam moment at the column face; V_{pZ} is the
218 panel zone shear force; SDR_b , SDR_{pZ} , and SDR_c are the beam, panel zone and column
219 contributions to the story drift ratio, respectively. Referring to Fig. 4(a), the CFE model predicts
220 the onset of local buckling fairly well under sagging (slab in compression) and hogging (slab in
221 tension) bending excursions. The predicted flexural strength and stiffness of the beam possesses
222 an outstanding agreement with the measured one up to 6% story drift ratio. Deviation from the test
223 results occurs in the last sagging excursion as ductile tearing initiated in the bottom flange of the
224 beam during the test. Figure 4(b) shows that the model marginally over-predicts the panel zone
225 deformations by about 10%. This is due to the slightly higher predicted beam moment. However,
226 the panel zone contribution to the story drift, in both the CFE model and test, does not exceed 1%.
227 Consequently, the slight deviation in the panel zone response does not practically influence the
228 energy dissipation capacity of the beam-to-column connection as shown in Fig. 4(d).

229 Figure 4(c) demonstrates a noteworthy agreement between the predicted and measured
230 decomposed deformation contributions to the story drift. Referring to Fig. 4(d), the same
231 observations hold true with regards to the accumulated energy dissipated by each component. Note
232 that the peak deformation in the panel zone, in both the CFE and the test, occurs at 3% SDR. At
233 6% SDR, the demand on the panel zone drops substantially as the beams experience flexural
234 strength degradation. Accordingly, the panel zone contribution at 6% SDR is negligible in both
235 the CFE and the test (0.03% and 0.15% respectively).

236 Parametric Study with Two-bay Sub-system Models

237 Having established confidence in the CFE modeling approach, the effects of the floor slab
238 continuity and framing action on the seismic performance of steel MRFs is assessed. The presence
239 of neighboring gravity frames is also expected to provide some degree of additional restraint on

240 the steel MRF (Donahue et al. 2017), thereby enhancing these effects. However, the influence of
241 gravity framing is not considered in this paper. Three two-bay sub-systems, summarized in Table
242 1, are considered herein: S_D with deep beams, S_I with beams of intermediate depth, and S_S with
243 shallow beams. The sub-systems cover a range of beam sizes employed in typical low to mid-rise
244 steel MRF buildings (Elkady and Lignos 2014, 2015a; Tartaglia et al. 2018; Tsitos et al. 2018).
245 The centerline span length, $L = 8992\text{mm}$ (29-1/2'), column height, $H = 3962\text{mm}$ (13'), maximum
246 unbraced length, $L_b = 1676\text{mm}$ (5-1/2''), and the slab dimensions correspond to those of
247 subassembly specimen SPEC3 (Zhang et al. 2004). In all three sub-systems, the member
248 slenderness ratio L_b/r_y , summarized in Table 1, is compliant with ANSI/AISC 341-16 (AISC
249 2016b) for special moment frames. The shear span-to-depth ratio L_o/d_b of specimen S_D does not
250 quite satisfy the ductility requirements of ANSI/AISC 341-16 (AISC 2016b). Nonetheless, the
251 moment-shear interaction was found to be insignificant, especially in the absence of gravity load.
252 This is consistent with available test data on composite connections (El Jisr et al. 2019). Since the
253 maximum shear force that can be transferred through the shear studs is governed by the capacity
254 of the concrete slab, the degree of composite action, as defined by ANSI/AISC 360-16 (AISC
255 2016c), is the same for all three sub-systems ($\eta \sim 20\%$). The columns are sized to remain elastic
256 (see strong-column-weak-beam (SCWB) ratio in Table 1), whereas the web panel zones are sized
257 to comply with ANSI/AISC 341-16 (AISC 2016b). Equal displacement was imposed at the top of
258 the columns. This loading technique assumes a rigid diaphragm for the floor slab above the
259 considered sub-system. The sub-systems are subjected to a cyclic symmetric lateral loading history
260 up to an SDR of 6% (SAC Joint Venture 1997). Sub-system S_D is also subjected to a collapse-
261 consistent protocol (Suzuki and Lignos 2019) to investigate the influence of loading history on the
262 sub-system cyclic performance. For each sub-system, the seismic behavior is compared with that
263 of the corresponding interior joint subassembly featuring simplified boundary conditions.
264 Particular emphasis is placed on the hysteretic behavior of the composite beams, panel zones, and
265 shear studs, the accumulated beam axial shortening and beam axial force demands.

266 ***Lateral Drift Demand Contributions***

267 The deformation demands at the interior and exterior joints of the sub-systems are examined.
268 Figure 5 depicts the decomposed deflection contributions to the story drift ratios of specimens S_D
269 and S_S at selected SDRs. In particular, the columns remain elastic, as intended, with minimal

270 contribution to the SDR. At the interior joint of S_D and at 4% lateral drift demand (Figs. 5(a and
271 b)), the panel zone contribution to the story drift is around 35%. This is considerably higher than
272 the panel zone contribution in the corresponding interior joint subassembly (see Fig. 4(c)). The
273 axial restraint in the sub-system delays the flexural strength degradation in the beams, thereby
274 increasing the panel zone shear demand. At 6% drift amplitude, when beam local buckling
275 becomes more evident and the inelastic deformations concentrate in the beam, the panel zone
276 contribution to the story drift decreases to about 10%. While a similar behavior is observed in S_S
277 (Figs. 5(c and d)), the panel zone contribution to the story drift remains appreciable (~25%) at 6%
278 drift amplitude. Flexural strength degradation in shallow beams is minimal as discussed in the
279 following section. Notably, exterior joints exhibit a distinct asymmetric behavior. Therefore, the
280 panel zone contribution to the story drift is dependent on the direction of lateral loading.
281 Particularly, the demand on the panel zone is higher when the framing beam is subjected to sagging
282 (Figs. 5(a and c)) compared to hogging bending (Figs. 5(b and d)). The reasons behind this
283 asymmetric demand are investigated more thoroughly in the subsequent sections. Referring to Fig.
284 5(c), the exterior joint panel zone contribution to the 6% story drift is nearly 40% despite being
285 designed with a resistance-to-demand ratio, $R_v/V_d = 1.9$ (R_v and V_d are defined in Table 1).
286 Moreover, at large lateral drift demands ($SDR \geq 4\%$), the exterior joint panel zones deform in one
287 loading direction although the lateral drift demand is symmetric as shown in Figs. 5(b and d). In
288 order to offset this negative contribution of the panel zone, the beam contribution to the story drift
289 at the exterior joints exceeds the imposed drift demand. The behavior is nearly identical at both
290 exterior joints of the sub-systems.

291 ***Beam Hysteretic Response***

292 Referring to Fig. 6, the hysteretic response of the west beam is obtained for all three
293 configurations and compared to that of the corresponding interior joint subassembly. For reference,
294 $M_{b,West,C1}$ and $M_{b,West,C2}$ are the west beam moments at the face of columns C1, and C2,
295 respectively. Referring to Fig. 6(a), the beam flexural strength degradation under sagging bending
296 occurs at a fairly slow rate, even at 6% lateral drift demand, in the interior joint of sub-system
297 S_D when compared to that of the corresponding subassembly. Table 2 shows that beams in the two-
298 bay sub-systems may experience up to 25% less flexural strength degradation, than those in
299 subassemblies under symmetric-cyclic lateral loading. This particularly applies to deep and slender

300 beams that are prone to local buckling (Lignos and Krawinkler 2011). This behavior is attributed
 301 to the restraint provided by the floor slab and adjacent columns against the beam axial shortening,
 302 which results in the straightening of the beam local buckles. The straightening effect is more
 303 evident under hogging than sagging bending. Figure 6(a) shows that under hogging bending, the
 304 strength degradation is only slightly lower in the sub-system than in the subassembly. The beam
 305 hysteretic response at the interior joint of sub-systems S_I and S_S shows minimal flexural strength
 306 degradation, similar to that of the corresponding subassemblies. The former has a low web
 307 slenderness ratio, $h_b/t_w = 31.3$, which delays the formation of web and flange local buckling at
 308 large inelastic cycles (Lignos and Krawinkler 2011). On the other hand, sub-system S_S consists
 309 of a shallow steel beam; as such, the slab contribution to the flexural resistance of the composite
 310 beam is higher than that in S_D and S_I . This results in a lower compressive stress in the top beam
 311 flange, thereby limiting local buckling under sagging bending.

312 Beam flexural strength degradation under hogging bending occurs as a result of the formation
 313 of large buckles in the bottom flange of the beam. In a sub-system, these buckles are straightened
 314 out during the sagging bending excursions due to the slab restraint against axial shortening. This
 315 agrees with earlier observations from physical testing of composite-steel MRFs (Cordova and
 316 Deierlein 2005). Furthermore, the beam and slab continuity at the interior joint augments this
 317 restraint (Cordova and Deierlein 2005; Herrera et al. 2008). Referring to Figs. 7(a and b), the
 318 buckled portions of the beam web and flanges experience notable straightening upon load reversal.
 319 As a result, pinching behavior, caused by an increase in the rotational stiffness of the composite
 320 beams, is observed in their hysteretic response (see Fig. 6). The axial restraint induces additional
 321 tensile axial forces ($F_{a,W}^{\pm}$ and $F_{a,E}^{\pm}$) and moments ($M_{a,W}^{\pm}$ and $M_{a,E}^{\pm}$) in the beams. The latter are
 322 caused by non-uniform buckling along the beam depth. On the other hand, beams in cruciform
 323 subassemblies are free to shorten at their ends due to the simplified boundary conditions, resulting
 324 in an “accordion” effect due to the build-up of local buckles. Figures 7(c and d) suggest that, in a
 325 subassembly, beam flanges that buckled experience minor straightening upon load reversal. A
 326 comparison between Figs. 7(a and b) and Figs. 7(c and d) reveals that the extent of bottom flange
 327 local buckling is closely akin in subassemblies and sub-systems. Hence, the rate of strength
 328 degradation under hogging bending is also expected to be cognate. This is not the case for sagging
 329 bending. First, the net tensile axial force acting on the beam is larger under sagging than hogging
 330 bending ($|F_{a,E/W}^+ + F_{b,E/W}^+| > |F_{a,E/W}^- + F_{b,E/W}^-|$). Second, the additional moment induced by non-

331 uniform buckling along the beam depth is lower under sagging than hogging bending ($|M_{a,E/W}^+| <$
332 $|M_{a,E/W}^-|$) due to the restraint provided by the slab to the top flange of the beam. Third, the rate of
333 stud degradation is lower in sub-systems when compared to that of subassemblies. The composite
334 action is maintained even at large lateral drift demands ($SDR \geq 4\%$), which alleviates the
335 compressive force near the top flange and enhances it near the bottom flange.

336 Referring to Figs. 6(b, d and f), the beam hysteretic response at the exterior joint of sub-systems
337 is fully asymmetric despite the fact that the imposed loading history is symmetric. Particularly, the
338 exterior joint beams experience flexural strength deterioration only under hogging bending. This
339 behavior is a consequence of the asymmetric demand on the exterior column web panel zone. The
340 mechanistic reason behind the observation above is explained in the next section.

341 Another consequence of the beam local buckling extenuation due to the slab restraint, is the
342 underestimation of the probable maximum moment in the beam, M_f , calculated as per ANSI/AISC
343 358-16 (AISC 2016a). Figure 6 suggests that although M_f is predicted fairly well for the
344 subassembly featuring deep beams, it is underestimated by about 25% in sub-system S_D . The delay
345 in local buckling in the beams results in additional cyclic hardening that does not occur in the
346 subassembly. Additionally, since the North American design practice typically employs deep
347 beams with a low degree of composite action, the slab contribution to M_f is ignored according to
348 ANSI/AISC 358-16 (AISC 2016a). Hence, the underestimation of M_f is larger in sub-systems with
349 shallower beams (S_I and S_S), where composite action is more pronounced. This issue is critical (a)
350 for sizing columns to remain elastic based on the SCWB ratio; and (b) for estimating the panel
351 zone shear demands. The implications of the latter are discussed in the next section.

352 ***Panel Zone Hysteretic Response***

353 The hysteretic response of the beam-to-column web panel zones is shown in Fig. 8 for both the
354 interior and exterior joints. Referring to Figs. 8(a, c and e), the interior joint panel zones of the
355 sub-systems experience more shear yielding than their subassembly counterparts. This is
356 particularly true for specimens S_D and S_S in which higher moments are attained in the beams
357 framing the joint. At 4% SDR, the shear distortion reaches $6\gamma_y$, $3.6\gamma_y$ and $5.7\gamma_y$ for specimens S_D ,
358 S_I and S_S , respectively (γ_y is the shear distortion at initial yielding as defined according to
359 ANSI/AISC 341-16 (AISC 2016b)). At exterior joints, the column web panel zone hysteresis

360 shows a distinct asymmetric response (see Figs. 8(b, d and f)). The shear distortion in the exterior
 361 joint panel zones at 4% SDR reaches $7.3\gamma_y$, $8.5\gamma_y$ and $5.2\gamma_y$ for specimens S_D , S_I and S_S ,
 362 respectively. Despite being designed for a maximum distortion of $4\gamma_y$ as per ANSI/AISC 341-16
 363 (AISC 2016b), the composite action and axial restraint provided by the floor slab cause additional
 364 inelastic shear distortion. This is not expected to cause premature fracture in view of recent
 365 experimental findings (Shin and Engelhardt 2013). Interestingly, El Jisr et al. (2019) found that in
 366 composite beam-to-column connections, panel zones can develop a total shear distortion of $10\gamma_y$
 367 without experiencing premature fracture within the beam-to-column connection at a 5% lateral
 368 drift demand. However, for tall buildings, the excessive distortion in the panel zones may become
 369 a concern when considering second-order effects. The panel zone shear resistance at a given
 370 inelastic shear distortion should be compared with the respective shear demand from the
 371 intersecting beams and columns to avoid the formation of soft story mechanisms that could
 372 increase the collapse risk due to P-delta effects.

373 The asymmetry observed in the hysteretic response of the exterior column web panel zones is
 374 explained through the development of three mechanisms. The first two are related to the
 375 asymmetric flexural demand in the beam framing the exterior joint. That is, the flexural demand
 376 in the beam is higher under sagging than under hogging bending.

377 Mechanism 1 is a direct consequence of the composite action in the beam. The sagging flexural
 378 resistance of the beam is enhanced (up to 80%) due to the composite action, while the hogging
 379 flexural resistance is also enhanced, but to a lesser degree (up to 40%) depending on the slab
 380 reinforcement (El Jisr et al. 2019). The factor α_1 , shown in Fig. 9, accounts for this enhancement:
 381 $\alpha_1^\pm = W_{pl,c}^\pm / W_{pl}$ and $\alpha_1^+ > \alpha_1^- > 1.0$ ($W_{pl,c}^\pm$ is the plastic modulus of the composite section under
 382 sagging ($W_{pl,c}^+$) or hogging ($W_{pl,c}^-$) bending, and W_{pl} is the plastic modulus of the bare steel cross-
 383 section with respect to its strong axis). Furthermore, the presence of the slab increases the depth
 384 of the region in the steel cross-section subjected to compressive stresses under hogging bending
 385 and decreases it under sagging bending. Hence, flexural strength degradation is hastened under
 386 hogging excursions, and delayed under sagging excursions. The factor β_1^\pm shown in Fig. 9,
 387 accounts for the phenomenon associated with the delay of local buckling under sagging bending
 388 (β_1^+), and the progression of local buckling under hogging bending (β_1^-) in the composite beam:

389 $\beta_1^+ > 1.0 > \beta_1^- > 0$. Mechanism 1 is more prominent in shallow beams ($d_b \leq 500\text{mm}$) where the
 390 effects of composite action are more pronounced compared to deep beams ($d_b \geq 700\text{mm}$).

391 Mechanism 2 involves the restraint that the slab provides to the top flange of the beam. The slab
 392 restraint delays the formation of local buckles, and hence the flexural strength degradation of the
 393 beam under sagging loading excursions. The flexural strength of the beam increases due to strain-
 394 hardening. The factor α_2 , shown in Fig. 9(a), accounts for the additional strain hardening in the
 395 beam due to the restraint provided by the slab on the top beam flange: $\alpha_2 > 1.0$ regardless of the
 396 beam depth. The extent of the slab restraint to the top flange is dependent on the orientation of the
 397 steel deck. Cordova and Deierlein (2005) reported a higher restraint when the steel deck is oriented
 398 parallel to the beam. However, this issue is outside the scope of the present paper.

399 Mechanism 3 is caused by the axial restraint provided by the slab and the adjacent columns.
 400 This restraint induces a moment, as well as a net tensile force in the composite beam. The tensile
 401 force is non-uniform across the beam depth. That is, the axial force is comprised of a tensile force
 402 in the beam and a compressive force in the slab. Figures 9(a and b) show an idealization of the
 403 panel zone shear demand induced by the axial force in the composite beam. A force couple is
 404 assumed to act on the top and bottom locations of the panel zone. The factor $\gamma^\pm > 0$ represents the
 405 fraction of the composite beam axial force, N_b , acting in compression on the top beam flange under
 406 sagging (γ^+) and hogging (γ^-) bending respectively. Accordingly, the axial force increases the
 407 shear demand on the panel zone for sagging excursions by $\gamma^+ N_b^+$ (see Eq. (1)) and decreases it for
 408 hogging excursions by $\gamma^- N_b^-$ (see Eq. (2)). Note that the d_{eff}^\pm defined in Fig. 9, is the effective
 409 depth of the panel zone as per the recommendations of Kim and Engelhardt (2002). Under hogging
 410 bending, d_{eff}^- is equal to the distance between the centroid of the beam flanges, whereas under
 411 sagging bending, d_{eff}^+ is equal to the distance between the centroid of the concrete section and
 412 that of the beam bottom flange. Mechanism 3 is prevalent in beams that develop a large axial force
 413 due to the axial restraint. Typically, these are deep beams ($d_b \geq 700\text{mm}$) with a low degree of
 414 composite action ($\eta < 50\%$) as will be explained in the following sections.

$$415 \quad V_{PZ}^+ = M_{b,max}^+ / d_{eff}^+ - V_T^+ + \gamma^+ N_b^+ \quad (1)$$

$$416 \quad V_{PZ}^- = M_{b,max}^- / d_{eff}^- - V_T^- - \gamma^- N_b^- \quad (2)$$

417 Mechanisms 1 and 2 appear to be the most dominant. This is based on findings from past
418 experiments on T-section subassemblies with composite floor slabs (Kim and Lee 2017; Yamada
419 et al. 2009). The tests showed a distinct ratcheting response in the web panel zones despite the
420 absence of axial restraint on the beam end. However, further studies should be conducted to
421 quantify the relative importance of each mechanism on the panel zone demand.

422 ***Beam Axial Shortening***

423 The phenomenon of beam axial shortening has been observed in subassembly tests in which the
424 steel beams are free to move axially at their ends (Civjan et al. 2001; FEMA 2000a; MacRae et al.
425 2013; Qi et al. 2018). Axial shortening occurs as local buckling builds up in the plastic hinge region
426 (Cordova and Deierlein 2005). However, in an actual building, the axial restraint provided by the
427 composite slab and adjacent columns is likely to limit this shortening. This is particularly true at
428 interior joints where the slab is continuous, and for composite slabs with the deck ribs placed
429 parallel to the steel girder (Civjan et al. 2001; Cordova and Deierlein 2005). Accordingly, the over-
430 simplified subassembly boundary conditions may lead to overestimation of the extent of a beam's
431 local buckling and subsequent axial shortening.

432 Figure 10a shows the definition of beam axial shortening, δ_x , within a steel MRF bay. Referring
433 to Fig. 10(b), subassembly S_D beam experiences an excessive axial shortening of 50mm at 6%
434 SDR. Top and bottom flange buckling mostly accumulate after 3% SDR, which leads to a rapid
435 progression of axial shortening. Figures 10(c and d) shows that subassembly S_I and S_S beams do
436 not shorten as much (6mm and 3mm at 6% SDR respectively). The former comprises a W21x122
437 beam with a fairly low $h_b/t_w = 31.3$; the latter consists of a shallow beam ($d_b = 409$ mm) in
438 which composite action is pronounced. In both specimens, the growth of local buckling across the
439 beam depth is insignificant. Hence, axial shortening is minimal. Beams in sub-systems shorten
440 much less compared to their subassembly counterparts. For instance, sub-system S_D beam shortens
441 by 7mm at 6% SDR while sub-systems S_I and S_S beams do not practically experience shortening.
442 The axial restraint provided by the composite slab and the adjacent columns alleviates the local
443 buckling in the anticipated dissipative zone of the steel beam.

444 It is worth mentioning that beams in sub-systems S_D and S_I experience fairly minor elongation
445 (up to 4mm). MacRae et al. (2013) attributed this elongation to the difference in the positions of

446 the neutral axes at the beam ends. Under sagging bending, the neutral axis moves upwards towards
447 the slab, whereas under hogging bending the neutral axis remains close to the beam centerline.
448 This difference in the neutral axis positions is particularly noticeable in shallow beams. As a result,
449 net centerline elongation results from tension yielding at the beam center near the sagging end,
450 and compression yielding near the hogging end. Furthermore, the asymmetric shear distortion in
451 the exterior joint panel zone exaggerates this net centerline elongation.

452 ***Beam Axial Force***

453 Lateral loads are transferred to the column through the floor slab via two load paths (Cordova
454 and Deierlein 2005; MacRae and Clifton 2015). The first one consists of direct bearing of the slab
455 on the column face and a direct compression strut to the back of the column flange. The second
456 load path involves the transfer of shear forces from the slab to the beam through friction and the
457 shear studs. The resulting axial force in the beam is transferred to the column through the beam-
458 to-column connection. In subassemblies, the beam axial force at the location of the assumed
459 inflection points is zero, increasing to its maximum value at the column face. In sub-systems, the
460 axial restraint provided by the floor slab and columns causes an additional axial force in the beam.
461 The magnitude of the beam axial force is not constant along the length of the beam and depends
462 on the extent of the axial restraint. The beam axial force is higher near interior joints, where the
463 axial restraint is higher, than near exterior joints. In sub-systems, unlike subassemblies, the axial
464 force at the beam inflection point location is not zero. Accordingly, the additional axial force
465 resulting from the axial restraint in sub-systems is quantified at the inflection point.

466 Figure 11 shows the axial forces developed in the three sub-systems at the west beam inflection
467 point. At 4% SDR, the peak normalized tensile force ratio in the steel beam at the location of the
468 inflection point N_s/N_{pl} ($N_{pl} = R_y F_{yb} A_g$ as defined in Fig. 11(a)) is 8%, 2% and 5% for sub-
469 systems S_D , S_I and S_S respectively. The tensile force ratio at 6% SDR is 16%, 8% and 9% for sub-
470 systems S_D , S_I and S_S respectively. These values are expected to be higher near the interior joint.
471 EN 1998-1 (CEN 2004a) states that the bending-axial force interaction in the steel beams may be
472 disregarded as long as $N_s/N_{pl} < 15\%$. The floor slab is restrained by the shear studs, friction at
473 the beam-slab interface and the columns. As the steel beam attempts to shorten due to the spread
474 of local buckling across its depth, a compressive force, N_c , is generated in the slab in conjunction

475 with the tensile force in the steel beam. The compressive force is transferred through shear in the
476 studs, friction, bearing of the slab on the column face and a direct compression strut. At large
477 lateral drift demands ($SDR \geq 4\%$), the studs lose their shear capacity, and the last two load paths
478 transfer the compressive force to the slab. This is particularly true for deep beams ($d_b \geq 700\text{mm}$)
479 with low degree of composite action ($\eta < 50\%$) as discussed in the next section.

480 The axial forces in the steel beam and slab are dependent on several parameters. These relate
481 to the extent of beam axial shortening experienced in the absence of axial restraint, as well as the
482 level of axial restraint. First and foremost, the magnitude of the tensile force in the steel beam is
483 dependent on the susceptibility of the steel beam to local buckling across its depth. Since all three
484 configurations studied herein are adequately braced laterally (see [Table 1](#)) and have nominally
485 identical material properties, the difference in local buckling initiation in the beams is mostly
486 governed by the beam's cross-section geometry. [Lignos and Krawinkler \(2011\)](#) found that h_b/t_w ,
487 in particular, largely influences local buckling initiation in intermediate to deep steel beams. The
488 maximum N_s/N_{pl} ratio increases with increasing h_b/t_w (see [Table 1](#)). This is observed in [Fig. 11](#)
489 where specimen S_D experiences the highest axial tensile force ratio.

490 Local buckling in shallow beams ($d_b \leq 500\text{mm}$) is localized in the lower portion of the beam
491 due to the slab restraint on the top flange of the beam. This is also true for beams with high degree
492 of composite action ($\eta > 80\%$). Moreover, in the above cases, a compatibility compressive force
493 occurs when the beam ends are pushed apart (see previous section). The compressive force
494 alleviates the tensile force in the beam. Axial restraint is provided by the slab and the columns. In
495 shallow beams, the axial restraint provided by the slab is relatively higher than that in deep beams.
496 This is because the relative slab in-plane stiffness-to-beam axial stiffness is higher in shallow
497 beams than in deep beams. On the other hand, the axial restraint provided by the columns is
498 dependent on their flexural stiffness. Since columns sizes are strongly influenced by the SCWB
499 ratio, the flexural stiffness of the columns normally increases with the beam depth. Therefore, the
500 axial restraint provided by the columns is expected to be high in deep beams. As mentioned earlier,
501 the level of axial restraint is higher at the interior joint than at the exterior joint due to (i) slab
502 continuity and (ii) potentially stiffer columns at the interior joints. In the configurations considered
503 in this paper, the same column cross-sections are used at the interior and exterior joints. Thus, we
504 postulate that the difference in the axial restraint should not be substantial. The magnitude of the
505 axial force in the beam also depends on its axial stiffness. However, for equal depth beams, a larger

506 axial stiffness implies a stockier section with a lower susceptibility to local buckling. Based on the
507 above, the major factor that determines the magnitude of N_s/N_{pl} is the susceptibility of the beam
508 to local buckling across its depth. The main controlling parameters in the examined cases are the
509 beam depth, d_b and the web local slenderness ratio, h_b/t_w .

510 *Shear Stud Hysteretic Response*

511 Seismic loads are transferred from the slab into the beam through shear in the stud connectors
512 and friction at the beam-slab interface. In composite beams with shear studs as the weak link, early
513 loss of composite action is likely to occur as a result of shear stud failure (Cordova and Deierlein
514 2005). Consequently, seismic loads are predominantly transferred to the column by bearing of the
515 slab on the column face and a direct compression strut. This can lead to severe damage in the slab
516 due to concrete spalling. Damage in the slab can be reduced if the integrity of the shear studs is
517 maintained. From a design perspective, the shear studs at the slab-beam interface should sustain
518 their load-carrying capacity. To this end, both ANSI/AISC 341-16 (AISC 2016b) and EN 1998-1
519 (CEN 2004a) recommend a 25% reduction in the design shear resistance of the studs. However,
520 past studies have shown that the performance of shear studs in composite steel MRFs is better than
521 anticipated (Cordova and Deierlein 2005). An assessment of the stud degradation behavior is
522 performed by obtaining the hysteretic stud shear-stud slip response in each of the composite beam
523 specimens subjected to cyclic loading.

524 Figure 12 shows the hysteretic response of the west beam shear stud nearest to the interior joint.
525 The shear studs in subassemblies S_D , S_I and S_S , lose their load carrying capacity in sagging
526 bending at 4%, 5% and 6% SDR respectively. At 4% lateral drift demands, the studs belonging to
527 subassemblies S_I and S_S exhibit satisfactory behavior with little or no degradation, whereas that of
528 specimen S_D fails. This is despite the fact that all three specimens have the same slab configuration,
529 number of studs and degree of composite action. The deeper the beam is, the higher the shear
530 demand; hence, the more degradation in the shear studs connecting the beam to the slab. In sub-
531 systems, the stud shear force degradation is lower than that in the corresponding subassembly.
532 Referring to Fig. 12(b), the shear stud in sub-system S_D loses its capacity at 5% SDR compared to
533 4% in the subassembly. Similarly, Figs. 12(c and d) depict that the shear studs of sub-system S_I
534 and S_S remain intact at the end of the analysis. In subassemblies, noticeable axial shortening in the
535 beams tends to pry the beam away from the slab. Consequently, the shear studs that restrain the

536 beam against axial shortening are subjected to an additional shear demand. The additional demand
537 increases the stud shear force in the sagging bending regions and reduces it in the hogging bending
538 regions. A higher rate of strength degradation is observed in studs belonging to subassemblies
539 when compared to those in sub-systems. Initially, the hysteretic behavior of the shear studs
540 coincides as shown in [Figure 12](#). Once beam axial shortening initiates, a discrepancy in the
541 behavior of the shear studs is observed.

542 In EN 1994-1-1 ([CEN 2004b](#)), ductile shear connectors are defined as the ones with a
543 characteristic deformation capacity, $\delta_{uk} = 6\text{mm}$ at 90% of the ultimate shear resistance ([Bärtschi](#)
544 [2005](#)). Hence, the headed shear studs, with which the non-linear springs are calibrated, are ductile
545 as per EN 1994-1-1 ([CEN 2004b](#)). The maximum stud slip demands at 4% SDR are 11mm, 8mm
546 and 2mm for sub-systems S_D , S_I and S_S respectively. Out of the three sub-systems, only the stud
547 slip demand in S_S , does not exceed the characteristic deformation capacity of ductile shear
548 connectors. However, the stud slip in all cases is within 6mm (4mm, 2mm at 1mm for S_D , S_I and
549 S_S respectively) at modest lateral drift demands (i.e., 2%) characteristic of a design-basis
550 earthquake corresponding to a probability of exceedance of 10% in 50 years. Additionally, a higher
551 degree of composite action would decrease the shear demand on the studs. Vis-à-vis the above
552 discussion, the general consensus is that for shallow to intermediate composite beams ($d_b \leq$
553 500mm) no reduction in the shear resistance of studs is imperative as long as ductile shear studs
554 are used. For deeper beams, a reduction in the shear resistance of studs is deemed reasonable.

555 ***Influence of Loading Protocol***

556 In the previous sections, the hysteretic behavior of the 2-bay sub-systems was examined under
557 a symmetric cyclic lateral-loading protocol. However, this protocol overestimates the seismic
558 demand in the frame and subsequently the cyclic deterioration of the beams ([FEMA 2009](#)), if limit
559 states associated with structural collapse are of interest. In that respect, collapse-consistent
560 protocols ([Krawinkler 2009](#); [Maison and Speicher 2016](#); [Suzuki and Lignos 2019](#)) are more
561 realistic for estimating seismic demands in structural components at limit states associated with
562 earthquake-induced collapse ([Lignos et al. 2011](#)). In order to further comprehend the differences
563 in the hysteretic behavior of sub-systems subjected to symmetric cyclic and collapse-consistent
564 protocols, sub-system S_D is subjected to a collapse consistent-loading protocol derived according
565 to [Suzuki and Lignos \(2019\)](#). The protocol consists of three phases and represents a near-fault

566 ground motion with a low probability of occurrence. Each phase includes a few inelastic cycles
567 followed by a large monotonic push. The asymmetric drift demand replicates the characteristic
568 “ratcheting” behavior of frame structures prior to collapse (Lignos et al. 2011).

569 Referring to Fig. 13(a), at 5% SDR, the west beam hogging moment ($M_{b,West,C2}$ at the face of
570 column C2) degrades by less than 5% under the collapse-consistent protocol. On the other hand,
571 the beam’s flexural strength degradation is more than 20% under the symmetric cyclic loading
572 protocol. Due to ratcheting of the frame, no degradation in the sagging moment occurs. Local
573 buckling is minor and is localized in the lower portion of the steel beam. This explains the
574 marginally lesser amount of axial shortening (4mm) experienced in the west beam under the
575 collapse-consistent protocol compared to the symmetric cyclic protocol (7mm) as depicted in Fig.
576 13(c). The beam experiences greater cyclic degradation under the symmetric cyclic loading
577 protocol than the collapse-consistent loading protocol due to the larger number of inelastic cycles
578 in the former. This agrees with prior observations from large- and full-scale physical testing
579 (Elkady and Lignos 2018b; Suzuki and Lignos 2015). Figure 13(b) shows that the peak panel zone
580 shear distortion is higher under the collapse-consistent protocol ($10\gamma_y$) than that observed under
581 the symmetric cyclic protocol ($6\gamma_y$). The ratcheting response is mostly attributed to the asymmetric
582 drift demand. Finally, the hysteretic behavior of the west shear stud nearest to the interior joint is
583 compared in Fig. 13(d). The stud loss of shear resistance occurs at 5% SDR, regardless of the
584 employed lateral loading protocol. Prior to 5% SDR, the seismic shear demand in the studs is
585 similar under both loading conditions. Furthermore, since the shear strength degradation due to
586 cyclic loading is not significant (see Fig. 2), the rate of degradation is nearly the same in both
587 cases.

588 **Limitations and Assumptions**

589 Considering the modeling assumptions and simplifications discussed herein, it is worth
590 highlighting the following limitations: (i) the CFE model is not capable of capturing fracture in
591 the beam-to-column connection and ductile tearing due to extensive local buckling; (ii) bond slip
592 between the reinforcement and concrete is not modeled explicitly; (iii) no separation is allowed
593 between the concrete slab and the steel deck; and (iv) concrete spalling due to crushing is not
594 explicitly considered. Despite these shortcomings, the modeling approach is deemed capable of

595 simulating the physical mechanisms associated with the slab restraint and the overall cyclic
596 behavior of connections in sub-systems and subassemblies.

597 **Conclusions**

598 This paper investigates the effects of the axial restraint provided by the slab and the columns
599 (frame continuity) on the hysteretic behavior of typical beam-to-column connections with a
600 composite floor slab. First, a detailed continuum finite element (CFE) model is proposed and
601 validated with available experimental data. The CFE model explicitly captures the interaction
602 between the slab and the beam, as well as the cyclic degradation of the shear stud connectors. Next,
603 the CFE approach is extended to model two-bay sub-systems with three different beam depths
604 representative of both North American and European design practice. The effects of the axial
605 restraint and framing action are examined by comparing the behavior of sub-systems with that of
606 the corresponding subassemblies. The major findings are summarized below:

- 607 • Qualitatively, the panel zone contribution to the story drift is higher in the sub-system interior
608 joints than in the corresponding cruciform subassembly joints. This is attributed to the lower
609 rate of the beams' flexural strength degradation. In the sub-system exterior joints, the panel
610 zone contribution to the story drift is dependent on the direction of loading: under sagging
611 excursions, the panel zone contribution to the story drift may reach up to 40%, despite the
612 panel zone design compliance to the ANSI/AISC-341-16 seismic provisions ([AISC 2016b](#)).
613 On the other hand, under hogging bending, the beam deformation dominates the lateral drift
614 demand.
- 615 • Under symmetric-cyclic lateral loading, beams in two-bay sub-systems may experience up to
616 25% less flexural strength degradation than their subassembly counterparts. This is particularly
617 evident in deep and slender beams. In sub-systems, the local buckles in the beams are
618 straightened due to the axial restraint provided by the floor slab and the columns. It is observed
619 that the straightening is more prominent under sagging bending than hogging bending. This
620 leads to the underestimation of the probable maximum moment M_f (by up to 25%), even in
621 deep beams where the flexural strength amplification due to composite action is fairly small.
622 This issue may be compelling for sizing columns and estimating the shear demand in panel
623 zones of capacity-designed steel MRFs.

- 624 • The interior joint panel zones in sub-systems experience up to 15% higher shear distortion than
625 their subassembly counterparts. Their hysteretic behavior is symmetric. On the other hand,
626 exterior joint panel zones in sub-systems exhibit a distinct asymmetric response due to the
627 different shear demands under sagging and hogging bending. The difference in shear demands
628 is attributed to three underlying mechanisms namely: (i) composite action, (ii) the slab restraint
629 against top flange local buckling; and (iii) the axial restraint provided by the slab and the
630 columns. The CFE analysis reveals that panel zones in sub-systems may experience a shear
631 distortion higher than the anticipated value for which they were designed (i.e., $4\gamma_y$).
632 Nonetheless, premature fracture due to panel zone shear distortion is not expected as the
633 maximum shear distortion is lower than $10\gamma_y$.
- 634 • Subassembly beams may experience severe axial shortening (up to 50mm at 6% SDR). The
635 degree of axial shortening is higher in deep beams with high web slenderness ratios close to
636 the current compactness limits of highly ductile members according to the ANSI/AISC 341-
637 16 seismic provisions (AISC 2016b). On the other hand, beam axial shortening observed in
638 sub-systems is considerably less (up to 7mm at 6% SDR) than that observed in subassemblies
639 (up to 50mm at 6% SDR). It is inferred that axial shortening is overestimated in subassembly
640 experiments commonly used in experimental earthquake engineering. In real buildings, beam
641 axial shortening is much lower, akin to that in sub-systems.
- 642 • Axial forces develop in composite beams as a consequence of the axial restraint. At the
643 inflection point, the axial tensile force in the steel beam's cross-section may reach slightly
644 higher than $15\%N_{pl}$ at 6% SDR. The tensile force magnitude is dependent on the susceptibility
645 of the beam to local buckling across its depth, as well as on the level of axial restraint. The
646 former is particularly high in deep and slender beams ($d_b \geq 700\text{mm}$) with low degree of
647 composite action ($\eta < 50\%$). The latter depends on the relative in-plane slab-to-beam axial
648 stiffness (higher in shallow beams) and the flexural stiffness of the columns (higher in deep
649 beam sub-systems). This issue should be examined in conjunction with the catenary action
650 imposed to the steel girders of a beam-to-column connection due to column axial shortening
651 (Elkady and Lignos 2018b; Suzuki and Lignos 2015).
- 652 • Comparisons between the hysteretic behavior of shear studs in sub-systems and subassemblies
653 suggest that the shear force degradation in the latter is higher than that of the former. This is
654 due to axial shortening in the beam that tends to pry the beam away from the slab. The CFE

655 models indicate that higher stud shear force degradation occurs in sub-systems with deep
 656 beams than in those with intermediate to shallow beams. However, at 2% lateral drift demands
 657 associated with a design-basis earthquake, the stud slip demand remains within the
 658 characteristic deformation capacity of ductile shear connectors (6mm) according to EN 1994-
 659 1-1 (CEN 2004b). At 4% lateral drift demand, the slip demand exceeded 6mm in all but the
 660 sub-system with shallow beams. For shallow beams or beams with high degree of composite
 661 action (i.e., above 80%), it seems reasonable to omit the 25% reduction in shear strength of the
 662 studs required in both ANSI/AISC 341-16 (AISC 2016b) and EN 1998-1 (CEN 2004a).

- 663 • The response of sub-systems under collapse-consistent lateral load protocols suggests that
 664 beam flexural strength deterioration and axial shortening is inconsequential compared to that
 665 under a symmetric loading history. Conversely, the panel zone shear distortion may reach $10\gamma_y$
 666 in exterior joints. The shear stud hysteretic behavior does not seem to be influenced by the
 667 employed loading history.

668 **Data Availability Statement**

669 The developed and validated high-fidelity continuum finite element model of composite-steel sub-
 670 system moment-resisting frames including a newly developed Fortran code (VUEL) that can be
 671 compiled with ABAQUS for modeling the cyclic behavior of shear studs are publicly available in
 672 GitHub (Link: https://github.com/eljjsr/IMK_Pinching_VUEL).

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677 **Notation**

The following symbols are used in this paper:

- A_g = cross-sectional area of the steel beam
- D^- = rate of cyclic deterioration of the shear stud when the slab is under tension
- D^+ = rate of cyclic deterioration of the shear stud when the slab is under compression
- d_b = depth of the steel beam
- d_{eff}^- = effective depth of the column web panel zone for framing beam under hogging bending

d_{eff}^+	=	effective depth of the column web panel zone for framing beam under sagging bending
$F_{a,E}^-$	=	tensile force in the steel beam east of the interior column due to axial restraint (hogging bending)
$F_{a,E}^+$	=	tensile force in the steel beam east of the interior column due to axial restraint (sagging bending)
$F_{a,W}^-$	=	tensile force in the steel beam west of the interior column due to axial restraint (hogging bending)
$F_{a,W}^+$	=	tensile force in the steel beam west of the interior column due to axial restraint (sagging bending)
$F_{b,E}^-$	=	tensile force in the steel beam east of the interior column due to composite action (hogging bending)
$F_{b,E}^+$	=	tensile force in the steel beam east of the interior column due to composite action (sagging bending)
$F_{b,W}^-$	=	tensile force in the steel beam west of the interior column due to composite action (hogging bending)
$F_{b,W}^+$	=	tensile force in the steel beam west of the interior column due to composite action (sagging bending)
F_{yb}	=	specified minimum yield stress of steel
f_c'	=	compressive strength of concrete
f_t	=	tensile strength of concrete
H	=	height of the column
h_b	=	fillet-to-fillet web depth of the beam
L	=	span length of the beam
L_b	=	maximum laterally unbraced length of the beam
L_o	=	shear span of the beam
$M_{a,E}^-$	=	moment in the steel beam east of the interior column due to axial restraint (hogging bending)
$M_{a,E}^+$	=	moment in the steel beam east of the interior column due to axial restraint (sagging bending)
$M_{a,W}^-$	=	moment in the steel beam west of the interior column due to axial restraint (hogging bending)
$M_{a,W}^+$	=	moment in the steel beam west of the interior column due to axial restraint (sagging bending)
M_b	=	beam moment at the column face
$M_{b,E}^-$	=	moment in the steel beam east of the interior column due to composite action (hogging bending)
$M_{b,E}^+$	=	moment in the steel beam east of the interior column due to composite action (sagging bending)
$M_{b,max}^-$	=	maximum beam moment at the column face (hogging bending)
$M_{b,max}^+$	=	maximum beam moment at the column face (sagging bending)
$M_{b,W}^-$	=	moment in the steel beam west of the interior column due to composite action (hogging bending)
$M_{b,W}^+$	=	moment in the steel beam west of the interior column due to composite action (sagging bending)
$M_{b,west,C1}$	=	west moment in the composite beam at column C1 face
$M_{b,west,C2}$	=	west moment in the composite beam at column C2 face

M_f	= probable maximum beam moment at the column face as per ANSI/AISC 358-16 (AISC 2016a)
N_b^-	= axial force in the composite beam due to axial restraint (hogging bending)
N_b^+	= axial force in the composite beam due to axial restraint (sagging bending)
N_c	= compressive force in the slab due to axial restraint
N_{pl}	= axial yield strength of the beam
N_s	= tensile force in the steel beam due to axial restraint
Q	= stud shear force
Q_u^-	= ultimate strength of the shear stud when the slab is under tension
Q_u^+	= ultimate strength of the shear stud when the slab is under compression
Q_y^-	= effective yield strength of the shear stud when the slab is under tension
Q_y^+	= effective yield strength of the shear stud when the slab is under compression
R_v	= column web panel zone inelastic shear strength as per ANSI/AISC 360-16 (AISC 2016c)
R_y	= ratio of the expected to the specified minimum yield stress of steel beam
r_y	= radius of gyration of the beam about its weak axis (y-axis)
S_D	= specimen with deep beams
SDR_b	= beam contribution to story drift ratio
SDR_c	= column contribution to story drift ratio
SDR_{PZ}	= panel zone contribution to story drift ratio
S_I	= specimen with beams of intermediate depth
S_S	= specimen with shallow beams
s	= stud slip
s_p^-	= pre-capping slip capacity of shear stud when slab is under tension
s_p^+	= pre-capping slip capacity of shear stud when slab is under compression
s_{pc}^-	= post-capping slip capacity of shear stud when slab is under tension
s_{pc}^+	= post-capping slip capacity of shear stud when slab is under compression
s_u^-	= ultimate slip capacity of shear stud when slab is under tension
s_u^+	= ultimate slip capacity of shear stud when slab is under compression
t_d	= thickness of doubler plate
t_{stable}	= size of the stable time increment in explicit dynamic analysis
t_w	= thickness of beam web
U_x	= displacement degree of freedom in x-direction
U_y	= displacement degree of freedom in y-direction
U_z	= displacement degree of freedom in z-direction
V_B^-	= shear force in the column below web panel zone (hogging bending)
V_B^+	= shear force in the column below web panel zone (sagging bending)
V_d	= column web panel zone shear demand as per ANSI/AISC 341-16 (AISC 2016b)
V_{PZ}	= column web panel zone shear demand

$V_{PZ,I}$	= interior column web panel zone shear demand
$V_{PZ,W}$	= west column web panel zone shear demand
V_T^-	= shear force in the column above web panel zone (hogging bending)
V_T^+	= shear force in the column above web panel zone (sagging bending)
W_{pl}	= plastic section modulus of the bare steel section about its strong axis
$W_{pl,c}^-$	= plastic section modulus of the composite beam section about its strong axis (hogging bending)
$W_{pl,c}^+$	= plastic section modulus of the composite beam section about its strong axis (sagging bending)
α_1^-	= factor that accounts for the enhancement of beam flexural resistance due to composite action (hogging bending)
α_1^+	= factor that accounts for the enhancement of beam flexural resistance due to composite action (sagging bending)
α_2	= factor that accounts for the additional strain hardening due to slab restraint on the top beam flange (sagging bending)
β_1^-	= factor that accounts for the progression of beam local buckling under hogging bending
β_1^+	= factor that accounts for the delay of beam local buckling under sagging bending
γ^-	= factor that represents the fraction of N_b^- acting on the top flange of the beam
γ^+	= factor that represents the fraction of N_b^+ acting on the top flange of the beam
γ_y	= shear distortion of the column web panel zone at initial yielding as per ANSI/AISC 341-16 (AISC 2016b)
δ_{uk}	= characteristic deformation capacity of ductile shear studs as per EN 1994-1-1 (CEN 2004b)
δ_x	= centerline axial shortening of the beam
η	= degree of composite action as per ANSI/AISC 360-16 (AISC 2016c)
κ_d	= parameter for the break point displacement due to pinching behavior in the stud
κ_f	= parameter for the break point force due to pinching behavior in the stud
λ_k	= parameter for stiffness deterioration of the shear stud under cyclic loading
λ_s	= parameter for strength deterioration of the shear stud under cyclic loading
μ	= steel-to-concrete coefficient of friction
ω_{max}	= highest element eigenfrequency in the CFE model

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920 **Table 1.** CFE virtual testing matrix

		Beam					Column		R_v / V_d		SCWB	
	Section	d_b [mm]	$\frac{h_b}{t_w}$	$\frac{b_f}{2t_f}$	$\frac{L_o}{d_b}$	$\frac{L_b}{r_y}$	Section	t_d [mm]	Int.	Ext.	Int.	Ext.
$S_D^{a,b}$	W36x150	911	51.9	6.4	4.5	26.7	W27x194	12.7	1.1	1.2	1.5	2.81
S_I^a	W21x122	551	31.3	6.5	7.6	22.6	W24x162	22.2	1.2	1.2	2.1	3.81
S_S^a	W16x45	409	41.1	6.2	10.5	42.0	W14x132	6.4	1.2	1.9	3.4	5.98

a = Cyclic symmetric loading history up to 6% story drift (SAC Joint Venture 1997)
 b = Collapse consistent loading protocol with two phases (Suzuki and Lignos 2019)
 R_v = Panel zone inelastic shear strength (AISC 2016c)
 V_d = Panel zone shear demand (AISC 2016b)
 $SCWB$ = Strong-column-weak-beam ratio (AISC 2016b)

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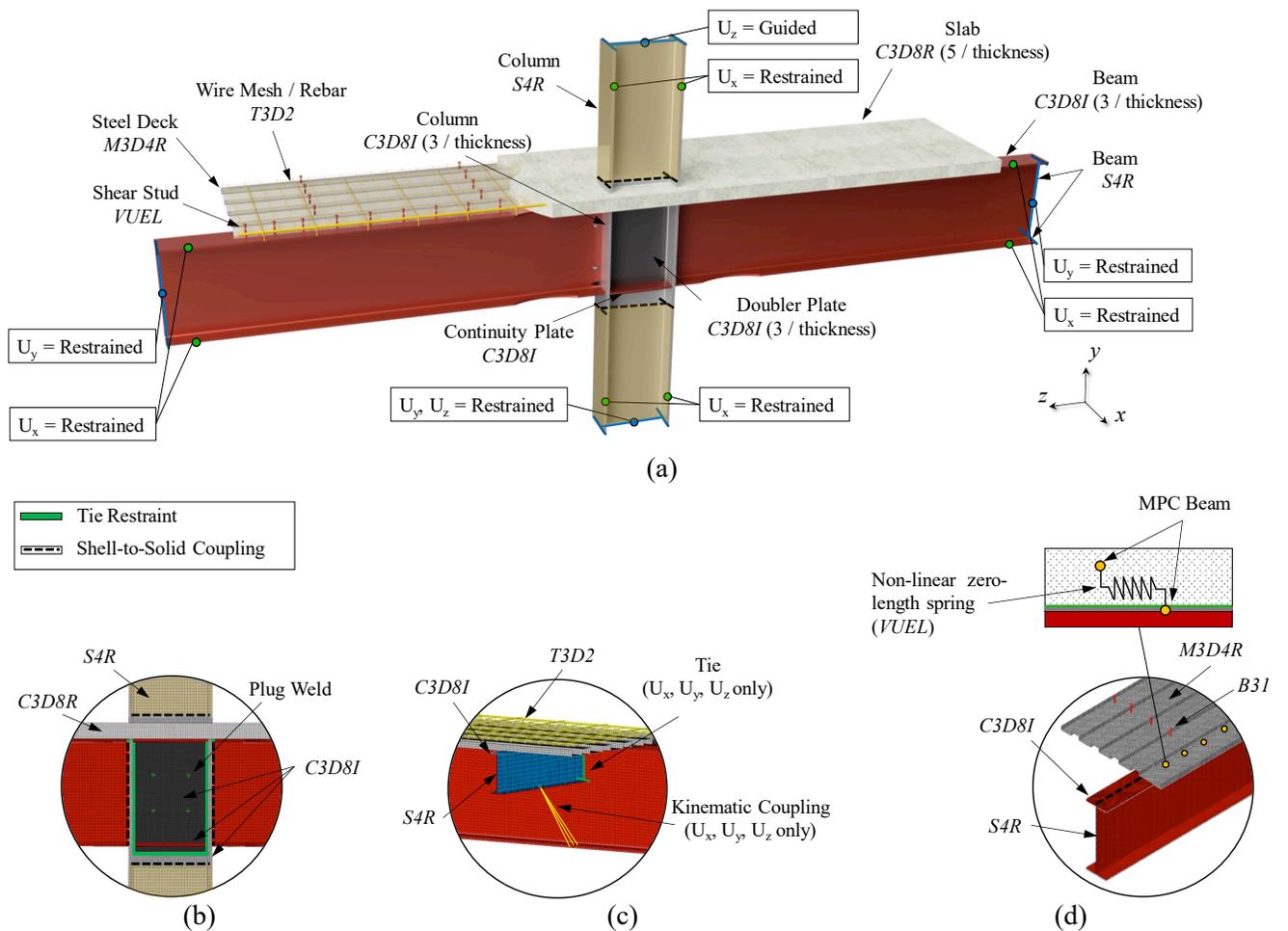
922

923 **Table 2.** Maximum west beam moment degradation at 6% SDR

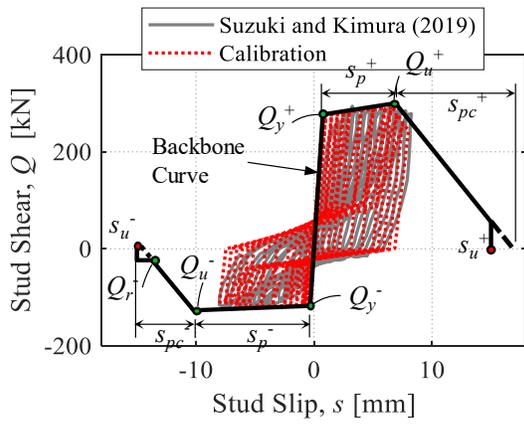
		Sagging Bending		Hogging Bending	
		<i>WI</i>	<i>WE</i>	<i>WI</i>	<i>WE</i>
S _D	Subassembly	34%	NA	44%	NA
	Sub-system	12%	4%	23%	38%
S _I	Subassembly	3%	NA	6%	NA
	Sub-system	0%	0%	6%	6%
S _S	Subassembly	16%	NA	9%	NA
	Sub-system	0%	0%	10%	19%

924

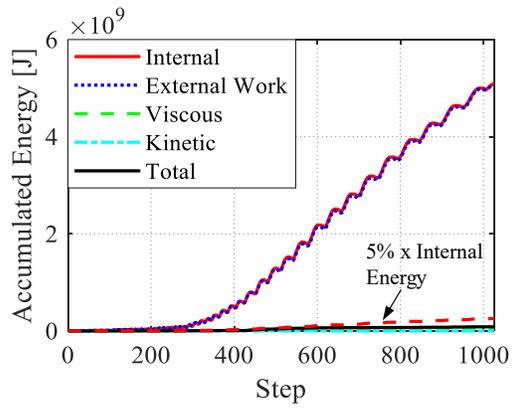
925



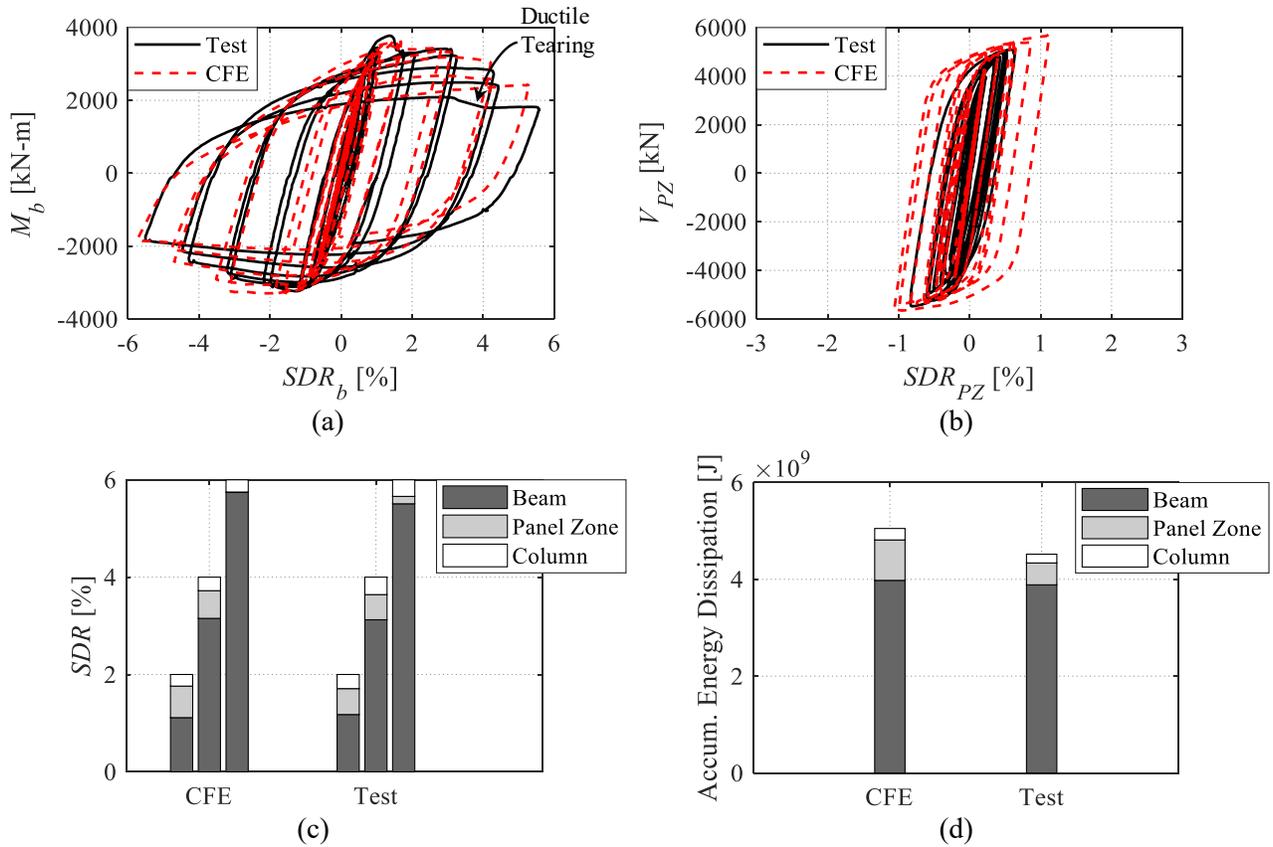
926 **Fig. 1.** (a) Continuum finite element model specifics for a typical subassembly configuration with
 927 composite floor slab (b) column web panel detail; (c) transverse beams detail; (d) shear studs detail



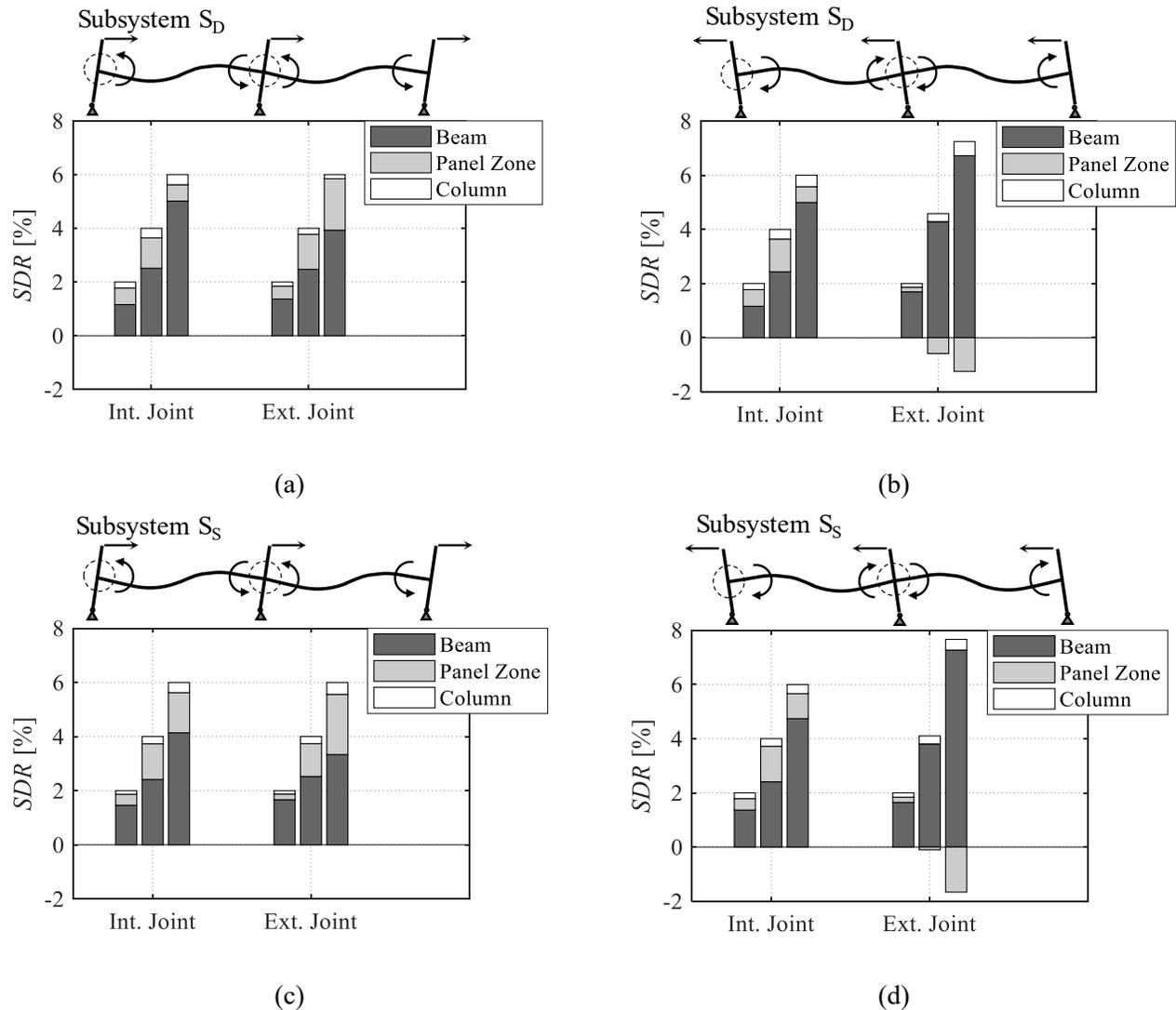
928 **Fig. 2.** Calibration example of a cluster of four 19mm cyclically-loaded shear studs (data values were
 929 reproduced from Suzuki and Kimura 2019)



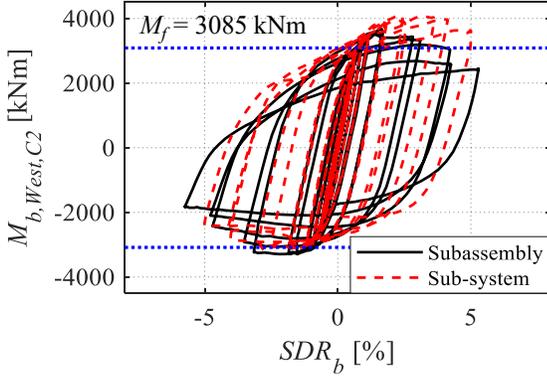
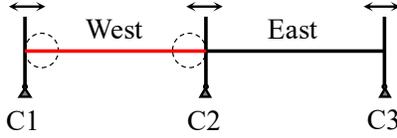
930 **Fig. 3.** Model energies accumulated throughout the explicit continuum finite element analysis



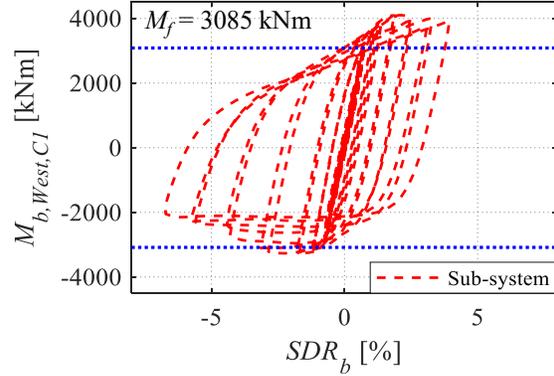
931 **Fig. 4.** Comparison between simulated and experimental results (experimental data are reproduced from
 932 [Zhang et al. 2004](#)) (a) beam hysteretic behavior; (b) panel zone hysteretic behavior; (c) component
 933 contribution to the story drift at the first cycle of 2%, 4% and 6% amplitudes; (d) accumulative energy
 934 dissipation in each component



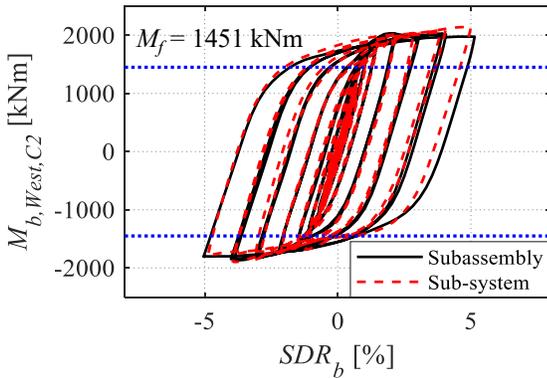
935 **Fig. 5.** Component contribution to the story drift at the 2%, 4% and 6% cycles; (a) subsystem S_D loading
 936 in the east direction; (b) subsystem S_D loading in the west direction; (c) subsystem S_S loading in the east
 937 direction; (d) subsystem S_S loading in the west direction



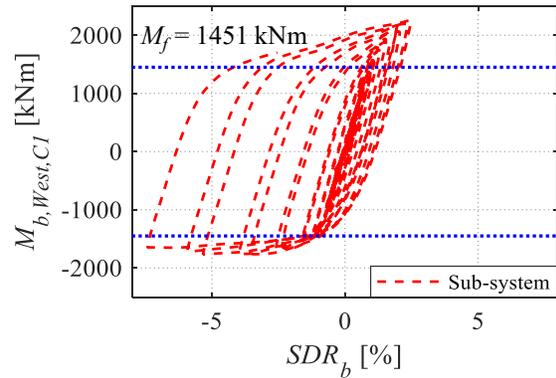
(a) W36x150, S_D Interior Joint



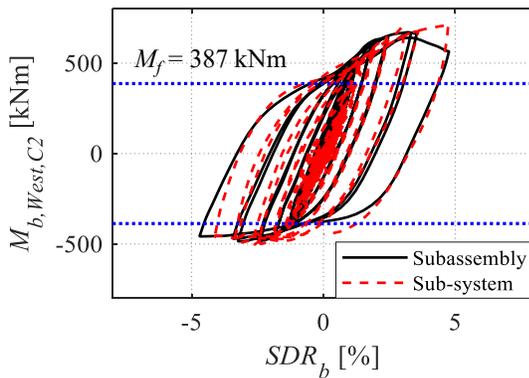
(b) W36x150, S_D Exterior Joint



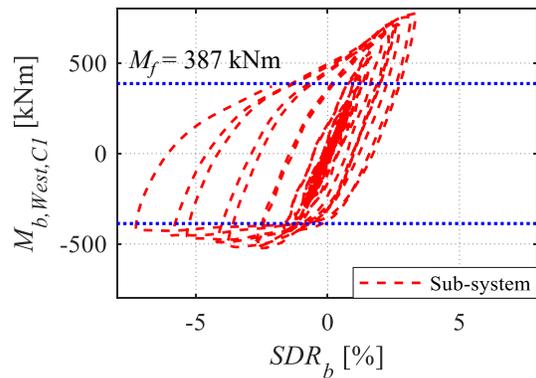
(c) W21x122, S_I Interior Joint



(d) W21x22, S_I Exterior Joint



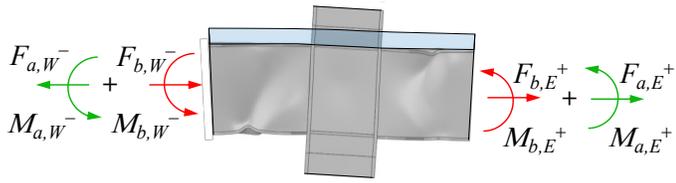
(e) W16x45, S_S Interior Joint



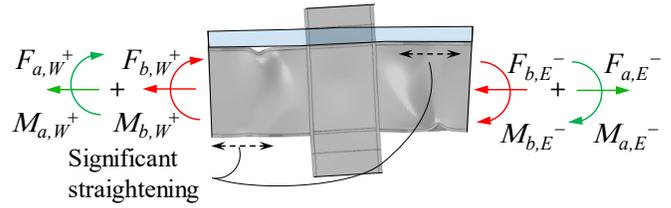
(f) W16x45, S_S Exterior Joint

938 **Fig. 6.** West beam hysteretic response - comparisons between sub-system and subassembly response (*left:*
939 interior joint; *right:* exterior joint)

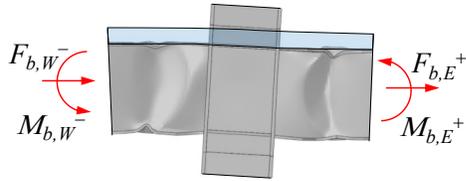
SDR
+ → -



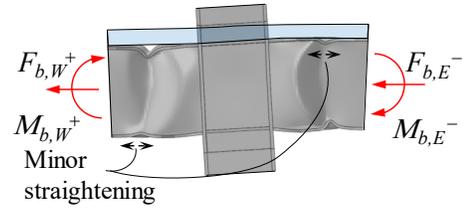
(a) Sub-system, Negative SDR



(b) Sub-system, Positive SDR

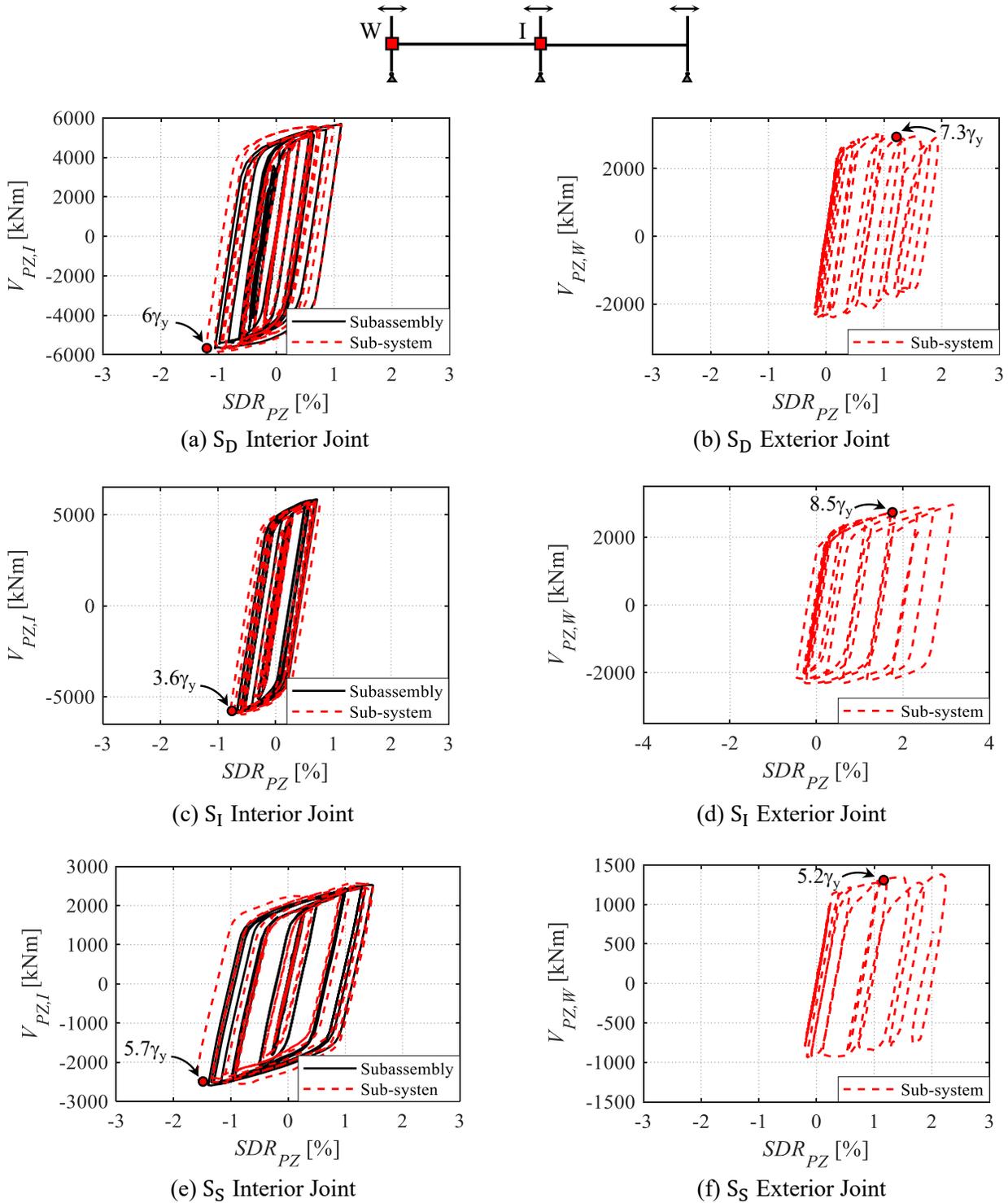


(c) Subassembly, Negative SDR



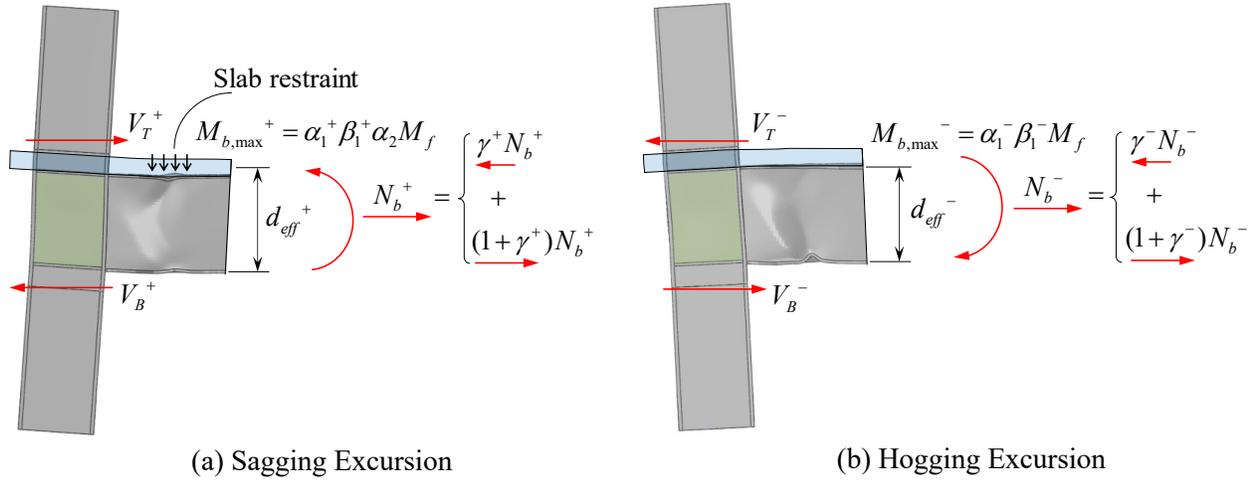
(d) Subassembly, Positive SDR

940 **Fig. 7.** Straightening mechanism of the buckles in the steel beam upon load reversal

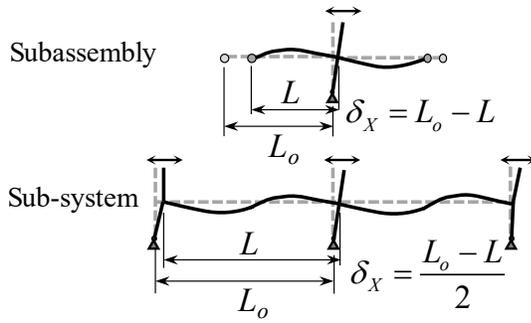


941 **Fig. 8.** Panel zone hysteretic response - comparisons between sub-system and subassembly response

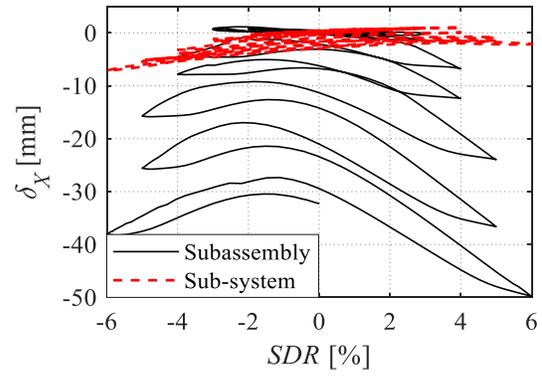
942 (*left: interior joint; right: exterior joint*)



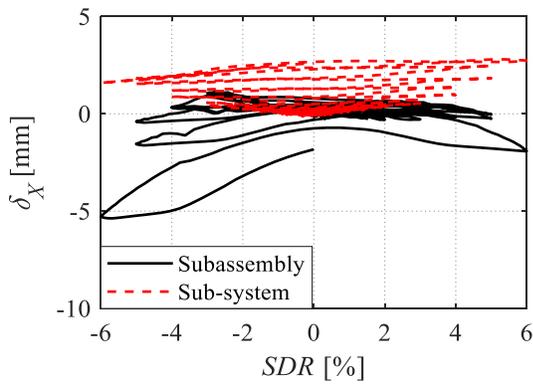
943 **Fig. 9.** Asymmetric shear demand on the exterior column web panel zone



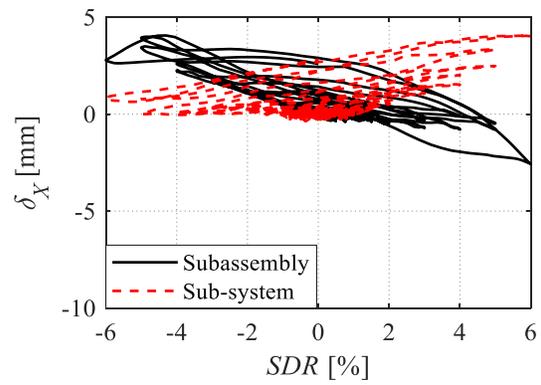
(a) Definition



(b) W36x150, S_D

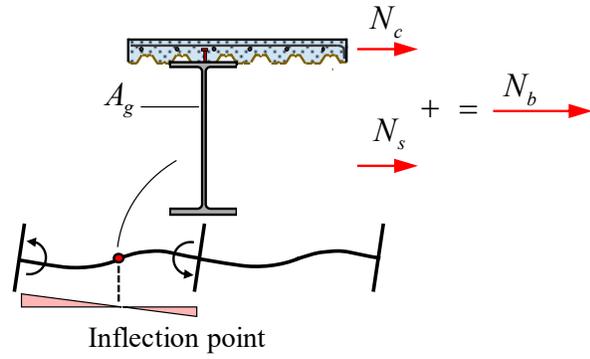


(c) W21x122, S_I



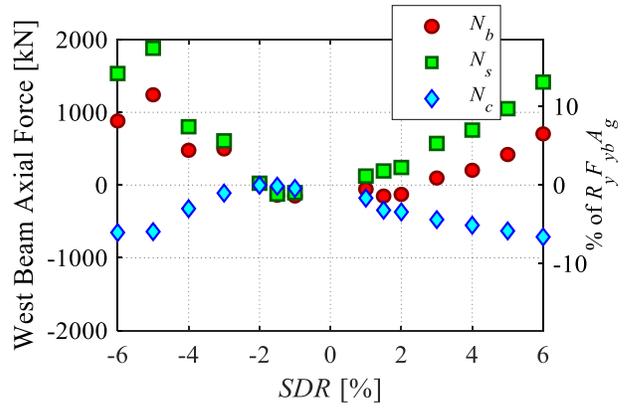
(d) W16x45, S_S

944 **Fig. 10.** West beam axial shortening – comparisons between sub-system and subassembly response

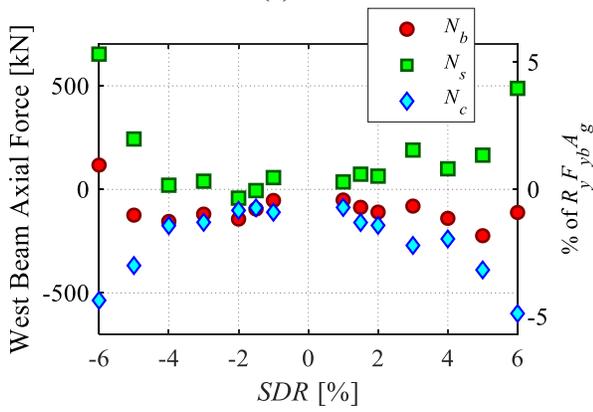


RF_{yb} is the expected yield strength of the steel beam

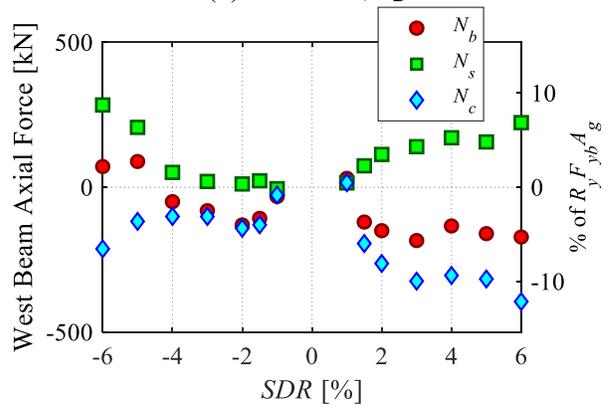
(a) Definition



(b) W36x150, S_D

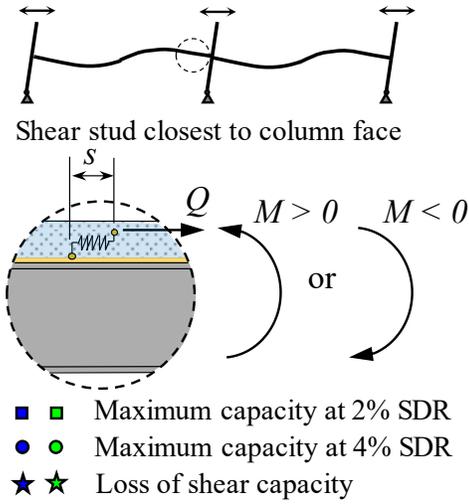


(c) W21x122, S_1

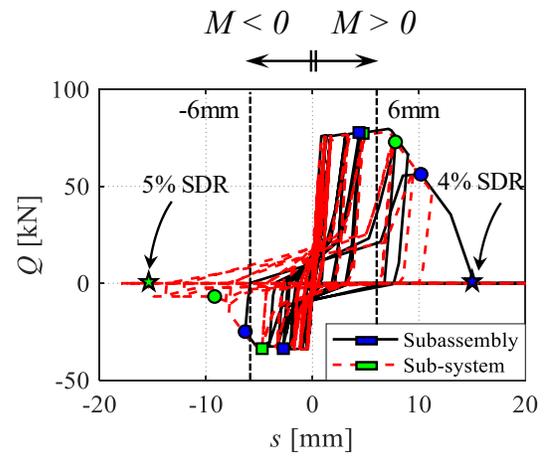


(d) W16x45, S_5

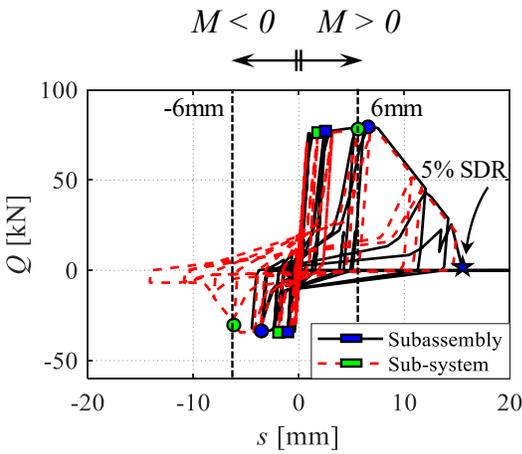
945 **Fig. 11.** Slab restraint-induced axial forces in the west beam of the sub-systems at peak story drifts



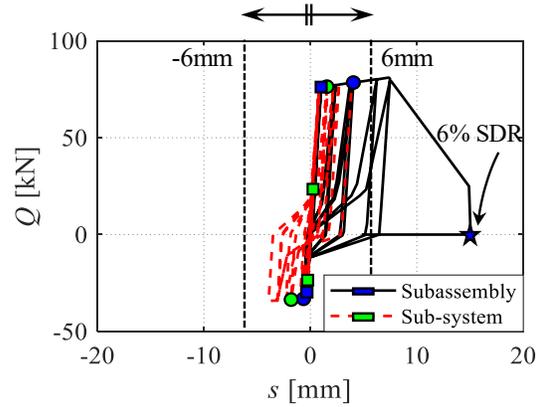
(a) Definition



(b) W36x150, S_D

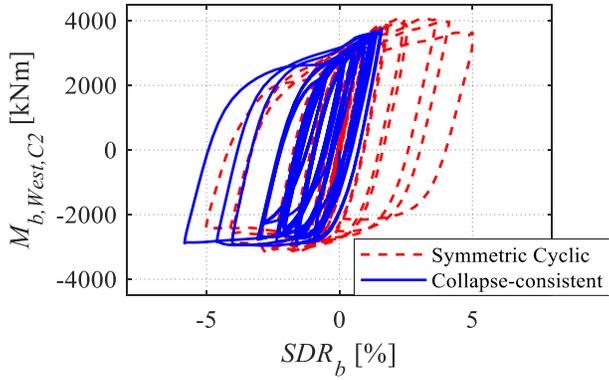
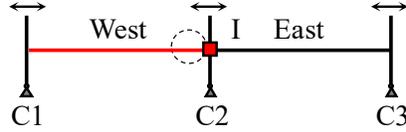


(c) W21x122, S_I

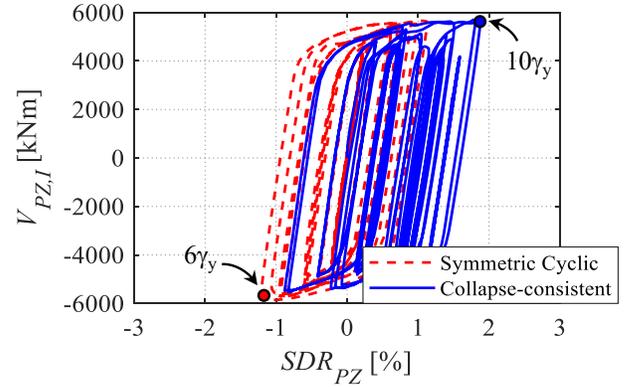


(d) W16x45, S_S

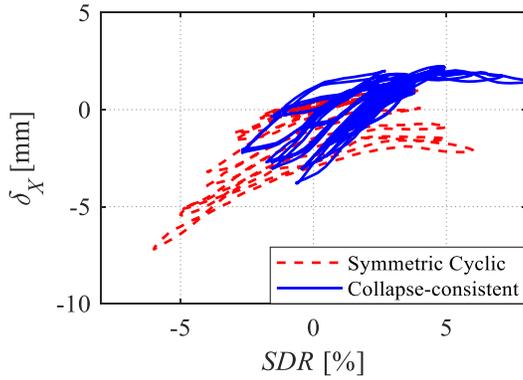
946 **Fig. 12.** West beam shear stud hysteretic response – comparisons between sub-system and subassembly
947 response



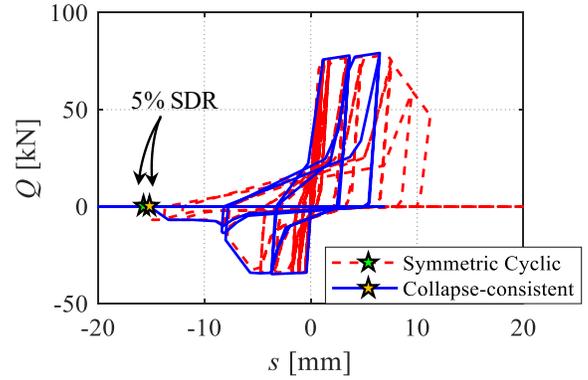
(a) West Beam Hysteretic Response



(b) Panel Zone Hysteretic Response



(c) West Beam Axial Shortening



(d) Shear Stud Hysteretic Response

948 **Fig. 13.** Sub-system S_D hysteretic response: comparisons between symmetric cyclic and collapse-
 949 consistent loading protocols

950