

Cyclic Behavior of Multi-Row Slit Shear Walls Made from Low Yield Point Steel

Liusheng He¹; Takuma Togo²; Kazuhiro Hayashi³; Masahiro Kurata, M.ASCE⁴; and Masayoshi Nakashima, M.ASCE⁵

Abstract: The steel slit shear wall has attracted much attention as a lateral force-resisting system. However, issues, such as fractures formed at the slit ends and pinched hysteresis, reduce energy dissipation. To address these issues, the authors have developed a steel slit shear wall made from low yield point steel that has a low yield stress and large ductility and strain hardening. Steel slit shear walls made from low yield point steel dissipated energy at small lateral drifts, shear deformation was evenly distributed among all rows, fracture was eliminated, and “fat” hysteresis without the requirement for out-of-plane constraints was feasible. By adjusting dimensions of the link (segment divided by slits) and the number of rows of links while maintaining the required shear strength and stiffness, a small width-to-thickness ratio for the links was achievable to ensure the in-plane behavior of links and thus good energy dissipation. The combined hardening model in ABAQUS simulated well the large strain hardening of low yield point steel. A proposed design procedure that ensures good energy dissipation was given.

Author Keywords: Steel slit shear wall; Low yield point steel; Strain hardening; Width-to-thickness ratio; Cyclic tests.

Introduction

Shear wall systems that use steel plates are common in earthquake engineering because of their large stiffness, lightness, and ductility. Among the many types of steel shear walls, the steel plate shear wall (SPSW) and steel slit shear wall (SSSW) are common in Japanese seismic

¹Research Fellow, Disaster Prevention Research Institute, Kyoto Univ., Gokasho, Uji, Kyoto, Japan (corresponding author).
Email: liush.he@gmail.com

²M.S. Candidate, Dept. of Arch. and Arch. Sys., Grad. Sch. of Eng., Kyoto Univ., Kyodai-Katsura, Kyoto, Japan.

³Assistant Professor, Dept. of Arch. and Civil Eng., Toyohashi Univ. of Technology, Toyohashi, Japan.

⁴Associate Professor, Disaster Prevention Research Institute, Kyoto Univ., Gokasho, Uji, Kyoto, Japan.

⁵Professor, Disaster Prevention Research Institute, Kyoto Univ., Gokasho, Uji, Kyoto, Japan.

22 design. The SPSW is accepted extensively in North America and is included in design standards
23 [e.g., AISC 2010; CSA 2009]. It resists shear deformation with tension field action after the
24 onset of buckling and presents substantial pinching behavior in its hysteresis loop [e.g., Roberts
25 and Ghomi (1991); Vian (2005); Berman and Bruneau (2005); Qu et al. (2008)]. The concept of
26 an SSSW is illustrated in Fig. 1. This wall is fabricated from steel plate and has a series of
27 rectangular segments (termed links) that are formed by laser-cutting vertical slits. When the wall
28 undergoes in-plane shear deformation (referred to as lateral drift), each link behaves as a flexural
29 member at the point of inflection located at mid-height. The yielding and hysteresis of links
30 become a source of energy dissipation similar to conventional steel hysteresis dampers
31 (Martinez-Rueda 2002). Since Hitaka and Matsui (2003) introduced the design philosophy of
32 SSSWs, many studies, including practical applications to real buildings, have been reported on
33 [e.g., Hitaka et al. (2007, 2009); Cortes and Liu (2011); Ke and Chen (2012)].

34 Energy dissipation of an SSSW is controlled by the link's width-to-thickness ratio (b/t) and
35 aspect ratio (l/b). Wider links (large b/t) can yield a greater strength and stiffness. However, if
36 the link is too wide in comparison with its thickness, local buckling of the link occurs and this
37 decreases the energy dissipation. Different means have been proposed to prevent link buckling
38 [e.g., Hitaka et al. (2009); Ma et al. (2010); Cortes and Liu (2011)]. However, improvements in
39 energy dissipation were mediocre and adverse effects resulted. Because links cannot buckle out
40 of plane to distribute the deformation with the buckling restraint, stress concentrations at the
41 ends of the link accelerate fracture formation, which may result in brittle failure. If the link is too
42 long (large l/b), it becomes too flexible and its energy dissipation is decreased significantly. For
43 relatively short links, the arrangement of links in multiple rows (m) can be used. However,
44 concentrated fracture on a single row may occur, as observed in the work of Cortes and Liu

45 (2011). Further investigation is required to eliminate fractures at the ends of the link, and
46 especially their concentration in a single row.

47 Low yield point steel (LYP), which has a low initial yield stress, large ductility, and exhibits
48 strain hardening, is a possible practical solution. SSSWs made from LYP enter the plastic stage
49 more rapidly than their surrounding frame. The large strain hardening of LYP enables plasticity
50 expansion over larger regions and thus dissipates more energy, and the large ductility of LYP
51 eliminates fracture at the ends of the link. When equal lateral bearing capacity is expected,
52 SSSWs made from LYP must be thicker relative to those made from conventional steel. The
53 increased thickness delays local buckling of individual links and thus there is a reduced need for
54 out-of-plane constraints. The application of LYP to shear walls is not new, and has been
55 presented in the work of Nakashima et al. (1995), Matteis et al. (2003), Chen and Jhang (2006),
56 and Zhang et al. (2012), but no work has been conducted on the application of such steel to slit
57 shear walls.

58 The authors propose an unstiffened SSSW design using LYP for improved energy
59 dissipation. Quasi-static cyclic testing on four 1/3-scaled SSSW specimens was conducted. A
60 series of comparative investigations were used to assess the performance of SSSWs made from
61 LYP and prediction of shear strength, stiffness, and hysteretic behavior.

62 **Strength and stiffness**

63 An SSSW must have sufficient strength to resist the lateral load and sufficient stiffness to
64 satisfy the design criterion on lateral drift. For an individual link, using the classical beam theory,
65 the shear force that corresponds to the first yielding at the ends of the link, termed the yield
66 strength $Q_{y.link}$, is:

$$67 \quad Q_{y.link} = \frac{2M_y}{l} = \frac{b^2 t}{3l} \sigma_y, \quad (1)$$

68 where $M_y = b^2 t / 6 \sigma_y$ is the moment at the end of the link when first yielding begins, b is the
 69 width of the link, l is the length of the link, t is the plate thickness, and σ_y is the yield stress.

70 Assuming that the plastic hinge is formed at the ends of the link where maximum strain
 71 occurs, the shear force that corresponds to full plasticity, termed the plastic strength $Q_{P.link}$, is:

$$72 \quad Q_{P.link} = 1.5 Q_{y.link} \quad (2)$$

73 For an SSSW with links in multiple rows (Fig. 1), the total yield/plastic strength is
 74 estimated by summation of all individual links:

$$75 \quad Q_y = \sum_{i=1}^n \frac{tb^2}{3l} \sigma_y = \frac{ntb^2}{3l} \sigma_y = \frac{Bt}{3\alpha} \sigma_y, \quad (3)$$

$$76 \quad Q_p = 1.5 Q_y, \quad (4)$$

77 where $B = bn$ is the width of the wall neglecting the width of slits and $\alpha = l / b$ is the aspect ratio
 78 of the link.

79 The elastic stiffness of the SSSW was proposed by Hitaka and Matsui (2003), and is
 80 calculated by summing all contributions from individual links and unslitted portions, as given in
 81 Eq. (5). Shear and flexural deformations of individual links are considered, and the contribution
 82 of an unslitted portion is accounted for through its shear deformation.

$$83 \quad K = \frac{1}{\frac{1.2(H - ml)}{GBt} + \gamma \frac{l^3}{Et b^3} \frac{m}{n} + \frac{1.2l}{Gbt} \frac{m}{n}} = \frac{Bt / H}{\frac{1.2}{G} + \gamma \frac{\beta \alpha^2}{E}}, \quad (5)$$

84 where E is Young's modulus; G is the shear modulus; 1.2 is the shear deformation shape factor
 85 for a rectangular section; $\gamma = (1 + 1 / \alpha)^3$ is a multiplier that reflects the flexibility at the ends of
 86 the flexural links, with $\gamma = 1$ denoting a perfectly rigid boundary and otherwise $\gamma > 1$; m is the

87 number of rows of links; n is the number of links in one row; H is the height of the wall; and
88 $\beta = ml / H$ is the slit fraction, which is the ratio of total link length to wall height.

89 A noteworthy feature of the SSSW is that the shear strength and stiffness can be designed
90 separately. Given the overall dimensions and material properties of the wall, the yield strength is
91 dependent solely on the link's aspect ratio (α) (Eq. (3)) and the stiffness is determined from α
92 and the slit fraction (β) (Eq. (5)), regardless of n and m . Therefore, the same values of α and β
93 guarantee the same yield strength and stiffness without a need to change the overall dimensions
94 and material properties of the wall, which provides flexibility in the design.

95 **Test preparation**

96 *Material properties*

97 Mild steel SS400, which is commonly used in Japan and has a specified minimum ultimate
98 strength of 400 MPa, and LYP100, which is an LYP with a nominal yield stress of 100 MPa,
99 were used. Compared with SS400, LYP100 has a lower initial yield stress and larger ductility
100 and strain hardening. These properties allow for early material yielding starting from a small
101 lateral drift and ensure good deformation capacity. Based on uniaxial tensile tests, SS400 has a
102 yield stress of 304 MPa and an ultimate stress of 431 MPa; LYP100 has a yield stress of 96 MPa
103 and an ultimate stress of 269 MPa. Fig. 2 shows the obtained stress-strain relationships.

104 A cyclic loading test of an SSSW made from LYP100, with four 90-mm-wide links (overall
105 dimensions of 360 mm \times 360 mm) using a 9-mm-thick plate (Fig. 3(a)), was used to calibrate the
106 material properties of LYP100 for numerical simulation. The solid lines of Fig. 3(b) show the
107 hysteretic curves obtained from the test.

108 In the numerical simulation, the commercial finite element code ABAQUS 6.10 (Dassault
109 Systèmes, 2004) was used, in which a three-dimensional four-node shell element with reduced

110 integration was used to represent the plate. The combined hardening model in ABAQUS was
111 used to model both the kinematic and isotropic hardening of LYP100. The kinematic hardening
112 property was determined from the results of the tensile coupon test. For the isotropic hardening,
113 a phenomenological curve fitting process was conducted, in which the input parameters were
114 adjusted until a reasonable match over most of the loading history was obtained. The numerical
115 results are plotted using dashed lines in Fig. 3(b). The numerical simulation was able to capture
116 the hysteretic behavior with reasonable accuracy.

117 ***Test specimen***

118 The prototype building where the SSSWs are installed is a medium-rise steel frame building
119 with a story height of 3500 mm and a span length of 5600 mm. The shear wall is proposed to
120 occupy the entire span with a yield strength of 800 kN. This strength is equivalent to a pair of
121 braces using a square hollow structural section of 175 mm × 175 mm × 6 mm made from A36
122 steel and placed in a chevron arrangement. Four 1/3-scaled SSSW specimens are designed with
123 overall specimen dimensions of 1150 mm × 1840 mm.

124 For a typical frame of a weak-beam strong-column type, a story drift of 0.5% is assumed to
125 be the elastic limit. Scaled-down specimens were designed to enter plasticity earlier than the
126 frame. Specimens 1–3, made from LYP100 with a plate thickness of 9 mm, were designed to
127 yield at 1/4 the elastic limit of the frame. Specimen 4, made from SS400 with a plate thickness of
128 4.2 mm, was designed to yield later at half the elastic limit of the frame. Link dimensions and the
129 number of rows of links in Specimens 1–3 (with links in two, three, and six rows, respectively)
130 were adjusted such that they had the same yield strength Q_y of 89 kN (estimated using Eq. (3))
131 and elastic stiffness K of 70 kN/mm (estimated using Eq. (5)). Specimen 4 with links in two rows
132 had a Q_y of 103 kN and a K of 36 kN/mm. The slightly smaller yield strengths of Specimens 1–

133 3 than Specimen 4 were designed with the consideration of a larger strain hardening of LYP100.
134 Four specimens have the same shear strength at a drift ratio of 1%, which will be presented in the
135 discussion of the test results. Table 1 summarizes the major parameters of each specimen.

136 Fig. 4 shows details of the four specimens. Vertical slits were made by a digitally controlled
137 laser cutting machine. Each specimen had top and bottom portions with round holes for
138 connection to the loading frame. The top and bottom portions were 90 mm and 100 mm deep,
139 respectively. Double-sided angles and high-strength bolts were used.

140 ***Test setup, instrumentation, and loading protocol***

141 The test setup was a portal frame with four pins at each corner, a height of 1748 mm, and a
142 column centerline spacing of 3000 mm, as shown in Fig. 5. The lateral deformation of the test
143 setup was controlled automatically using the loading system, which consists of a horizontal
144 hydraulic jack, a hydraulic pump system, and a control computer. The main components of the
145 test bed were: (a) top and bottom H-400 × 400 × 13 × 21 beams, (b) two H-250 × 250 × 9 × 16
146 columns, (c) four pin subassemblies, and (d) a fixed support for the actuator loading. The
147 deformation of the test setup was restrained to remain in plane using out-of-plane restrainers and
148 guiding beams.

149 The lateral drift applied to the shear wall was controlled by the jack's displacement. The net
150 shear deformation into the links was measured by attaching two displacement transducers at the
151 ends of the link in certain rows, as shown in Fig. 5(c). For Specimens 1, 2, and 4, the net shear
152 deformation into the links in all rows was measured. For Specimen 3 with six rows, the net shear
153 deformation of links was measured in the second, fourth, and sixth row from the top. Out-of-
154 plane deformation at the vertical center of the wall edge (D5 in Fig. 5(c)) was measured to detect
155 the onset of plate buckling.

156 The incremental two-cycle loading, which was adopted often in the experimental study of
157 shear walls, was used as shown in Fig. 6. At lateral drifts smaller than a drift ratio of 2%, the
158 incremental amplitude was a drift ratio of 0.25%. At lateral drifts larger than a drift ratio of 2%,
159 the incremental amplitude was a drift ratio of 0.5%. The same amplitudes were repeated twice to
160 a maximum drift ratio of 4%.

161 **Discussion of test results**

162 *Yield strength*

163 Yielding and plastification developed at the ends of the link is a unique feature of the SSSW
164 system. To observe the first yielding, foil strain gauges for measuring the elastic strains were
165 glued on the center links in each row, and on the front face of the links at their ends, as shown in
166 Fig. 5(c). The gauges were glued in the longitudinal direction of the link and 3 mm inside its
167 edge (Fig. 5(d)).

168 The yield strength was determined as the shear force applied to the specimen when one of
169 the strain gauges exceeded the yield strain obtained from the coupon test. The yield strengths
170 obtained are listed in Table 2, together with the analytical strength estimated using Eq. 3. The
171 experimental and analytical strengths were similar, with an error of 16% for Specimens 3 and
172 errors less than 6% for the others.

173 As mentioned previously, four specimens were designed to have the same shear strength at
174 a drift ratio of 1%. The strengths at a drift ratio of 1%, $Q_{1\%,t}$, are listed in Table 2 and were
175 similar for all specimens, with a variance within 4%.

176 *Maximum strength*

177 The maximum strength obtained from the test is listed in Table 2. Here, the maximum
178 strength $Q_{\max,t}$ was defined as the largest absolute strength obtained to completion of the loading

179 with a drift ratio of 4%. The maximum strength was 25% larger than the plastic strength Q_p
180 (estimated using Eq. 4) for Specimen 4 made from SS400. For Specimens 1–3 made from
181 LYP100, the maximum strength was 2.1–2.4 times Q_p . The significant overstrength relative to
182 Q_p , especially for SSSWs made from LYP100, is ascribed to the combined effects of strain
183 hardening and tension fields formed at large drift ratios, which will be explained more in a later
184 section. To estimate the maximum strength in the design of beams and connecting angles where
185 the SSSW is installed, an amplification of the plastic strength should be considered cautiously.
186 For SSSWs made from LYP100 with a maximum drift ratio of 4%, 2.5 times Q_p can be used in
187 the estimation of maximum strength.

188 ***Initial stiffness***

189 The initial stiffness obtained from the test is also listed in Table 2. The predicted elastic
190 stiffness (K) agreed relatively well with the test results (K_t), with an error of less than 2%.
191 Specimen 4 made from SS400 entered into plasticity at a drift ratio of approximately twice that
192 of Specimens 1–3 made from LYP100 as designed. The yielding sequence, yielding of the slit
193 wall prior to the frame, was well controlled. The very similar stiffness (and shear strength) of
194 Specimens 1–3, with different link dimensions and number of rows of links, demonstrated the
195 flexibility in the SSSW design.

196 ***Shear deformation distribution among link rows***

197 The net shear deformation into links was measured by the relative displacement between the
198 two ends of the link in the same row. Taking Specimen 1 in Fig. 5(c) as an example, the
199 difference between D1 and D2 gives the net shear deformation for the links in the first row and
200 the difference between D3 and D4 gives the net shear deformation for the links in the second row.
201 Fig. 7 shows the shear deformation distribution among rows. Within a story drift ratio of 1%, the

202 net shear deformation for the links in Specimens 1–3 made from LYP100 was almost the same
203 among all rows. For Specimen 4 made from SS400 (Fig. 7(a)), the net shear deformation
204 measured on the first row was larger. This difference is caused primarily by the different nature
205 of the two grades of steel. Under in-plane shear deformation, the initial imperfection in the plate,
206 including geometrical imperfections such as non-uniform thickness or material defects such as
207 cracks and vacancies, triggered a slightly different allocation of shear deformation among the
208 rows. The large strain hardening of the LYP100 allowed the stress to increase with the
209 development of plasticity, which adaptively adjusted the shear strength of each row and
210 prevented the formation of a weak row with larger deformation. On the contrary, because of the
211 limited strain hardening of SS400 used in Specimen 4, the increase in stress at the row that
212 experienced a larger deformation was insufficient to transform it into a stronger row and thus the
213 uneven deformation was maintained.

214 ***Local buckling and fracture of individual links***

215 In the thin plate theory, the width-to-thickness ratio controls local buckling (Timoshenko
216 and Gere, 1961). For individual links with a suitable link height, the width-to-thickness ratio of
217 the link (b/t) controls the inelastic behavior. The early local buckling of links with a large b/t
218 reduces the energy dissipation. To ensure the in-plane behavior of links, a small enough b/t is
219 needed, which can be realized by cutting more slits into the wall and accordingly having more
220 narrow links (n). To maintain the same shear strength and stiffness, shorter links should be
221 arranged in more rows (m). Taking Specimens 1–3 made from LYP100 for example, the same
222 aspect ratio of the link ($\alpha = l/b$ of 5.9) and slit fraction ($\beta = ml/H$ of 0.7) provided the same
223 strength and stiffness, whereas the values of b/t are 7.8, 5.1, and 2.5, with links arranged in two,
224 three, and six rows, respectively. Notable local buckling of links in Specimen 1 occurred at a

225 drift ratio of 3.5%, but this behavior was not observed in Specimens 2–3 because of their smaller
226 values of b/t . With the steel of LYP100 used in this study, a b/t of 5.1 in Specimen 2 is
227 considered small enough to ensure the in-plane behavior of links. With a b/t of 7.8, which is the
228 same as that of Specimen 1, Specimen 4 made from SS400 exhibited local buckling of links at an
229 earlier drift ratio of 2%, together with the initiation of fracture at the ends of the link because of
230 the reduced ductility of SS400. Because Specimens 1 and 4 had approximately the same shear
231 strengths, the greater thickness of Specimen 1 (9 mm, and 4.2 mm for Specimen 4) resulted in a
232 low stress level, which explained the delayed local buckling of links in Specimen 1.

233 Fig. 8 shows the deformed specimens. Of Specimens 1–3 made from LYP100, only
234 Specimen 1 exhibited notable local buckling of links and no sign of fracture throughout the
235 loading in all specimens. For Specimen 4 made from SS400, local buckling and fracture at the
236 ends of the link occurred simultaneously at a drift ratio of 2% and finally penetrated a majority
237 of links in the second row after completion of the first cycle of a drift ratio of 3.2%.

238 *Effect of slit fraction*

239 In the direction of the wall's height, the proportion of individual links, represented by the
240 slit fraction $\beta = ml / H$, is also an important factor. A small β indicates short individual links
241 and a large unslitted portion. Because of the height difference of the link and the wall, short links
242 have a high strain level at the ends of the link subjected to the same lateral drift, which would
243 cause early local buckling of links. The behavior of the wall is dominated more by the plate
244 rather than by the individual links because of a large unslitted portion.

245 As shown in Fig. 5(c), the out-of-plane deformation at the wall edge (D5) was measured by
246 a displacement transducer to detect the onset of plate buckling. Specimens 1–3, with a β of 0.7,
247 showed a similar out-of-plane deformation. For illustration, only Specimen 1 is discussed here.

248 The out-of-plane deformation of Specimen 1 was observed at a drift ratio of 1%, with a
249 normalized deformation of 1/280 (defined as the absolute out-of-plane displacement divided by
250 the wall height). For Specimen 4, with a β of 0.42, the out-of-plane deformation occurred at a
251 drift ratio of 0.25%, with a normalized out-of-plane deformation of 1/300. This much earlier
252 onset of out-of-plane deformation of Specimen 4 was believed to result from the small β of 0.42,
253 which promoted the overall plate behavior more significantly than the behavior of individual
254 links. With the increase in drift ratios, the out-of-plane deformation of all specimens increased.
255 At a drift ratio of 1%, the normalized out-of-plane deformation of Specimen 4 was 1/110 and
256 remained approximately constant for further loading. The concentration of deformation in a
257 lower link was a likely source for this behavior. Specimen 1 sustained a steadily increasing out-
258 of-deformation for larger drifts, but the normalized out-of-plane deformation remained 1/35 at a
259 drift ratio of 2%. (After 2%, the displacement transducer was removed to prevent it from being
260 damaged in case the deformation were beyond its range.) Considering that the middle part of the
261 wall had a much smaller out-of-plane deformation than the wall edge and no deterioration of the
262 shear strength appeared in the overall hysteretic behavior, the effect of out-of-plane deformation
263 was considered at most secondary.

264 Conversely, a large β yields a short unslitted portion between the rows of links. Yielding
265 that developed at the ends of the link is likely to expand over this portion, and if the unslitted
266 portion is too short, the assumption of a stiff boundary between the links will no longer be valid,
267 which also limits further development of plasticity at the ends of the link and thus lessens the
268 energy dissipation. The value of β is recommended as 0.65–0.85 by McCloskey (2006) to
269 ensure that the wall behavior is controlled by the links. A β of 0.7 for Specimens 1–3 follows
270 this recommendation.

271 ***Energy dissipation behavior***

272 The solid lines of Fig. 9 show the hysteretic curves, with the ordinate being the shear force
273 normalized by the plastic strength (Q_p). Within a drift ratio of 1%, the effect of large strain
274 hardening of LYP100 was demonstrated clearly by the isotropic “fatness” of the hysteretic loops.
275 Beyond a drift ratio of 1%, the strength increment slowed down but continued to grow with the
276 increase in lateral drift because of the formation of tension fields. Local buckling of links in
277 Specimen 1 caused slight pinching in the hysteresis starting from a drift ratio of 3.5%. Local
278 buckling of links did not occur in Specimens 2–3 and “fat” hysteretic loops resulted. For
279 Specimen 4 made from SS400, the shear strength degraded significantly after fracture occurred
280 at the ends of the link at a drift ratio of 2%. By using the steel of LYP100 with large strain
281 hardening, an out-of-plane constraint is not required to obtain “fat” hysteretic loops.

282 The equivalent damping ratios estimated using the standard procedure (Chopra, 2000) are
283 plotted for each drift ratio in Fig. 10, in which one loop in the first cycle was used for the
284 calculation. The equivalent damping ratios of Specimens 1–3 made from LYP100 was
285 approximately two times that of Specimen 4 made from SS400 before a drift ratio of 1%.
286 Beyond a drift ratio of 1%, the equivalent damping ratios of Specimens 1–3 were almost constant
287 with a ratio of 0.4 until completion of loading with a drift ratio of 4%.

288 Fig. 11 shows the energy dissipation, estimated as the summation of energy dissipated in the
289 first cycle at each drift ratio. Specimens 1–3 made from LYP100 dissipated nearly the same
290 energy to a drift ratio of 3.5%. At a drift ratio of 0.25%, the energy dissipation of Specimens 1–3
291 was 6.6 times that of Specimen 4, until the completion of loading of Specimen 4 at a drift ratio of
292 3.2%. The energy dissipation of Specimens 1–3 was no less than 1.7 times that of Specimen 4.
293 After 3.5% with further local buckling of links of Specimen 1, the energy dissipated decreased

294 by 13% at a drift ratio of 4%, compared with Specimens 2–3 where local buckling of individual
295 links did not occur.

296 ***Numerical analysis***

297 The hysteretic curves in dashed lines in Fig. 9 show the numerical results obtained from
298 ABAQUS. For Specimens 1–3 made from LYP100, the material properties calibrated in advance
299 (Fig. 3) were adopted and the simulation predicted relatively well the maximum strength and
300 hysteretic curve. For Specimen 4 made from SS400 with limited strain hardening, a simple
301 bilinear model was used, with a yield stress of 304 MPa and strain hardening of 1.1%
302 approximated from the tensile coupon test. The simulation agreed well with test results before
303 fracture occurred at a drift ratio of 2%.

304 **Design procedure**

305 Fig. 12 shows the proposed design procedure for the SSSW. Overall dimensions of the wall
306 (width B and height H) are determined based on architectural dimensions. The demands of yield
307 strength Q_y and stiffness K are determined by the expected lateral load distribution and addition
308 of stiffness for each story, respectively. Because a smaller aspect ratio of the link ($\alpha = l/b$)
309 gives larger Q_y and K values, a relatively small α should be used. Considering the balance
310 between the dimensions and strength desired in design, feasible values for α are 3–10.
311 According to Eq. (3), a proper plate thickness t can be chosen to satisfy the required Q_y ; then,
312 according to Eq. (5), K can be met by assigning a certain value for the slit fraction ($\beta = ml/H$).
313 β should be between 0.65 and 0.85 to ensure that individual links control the wall behavior. A
314 suitable β can be obtained by adjusting either α or t , or both. After the determination of β , the
315 number of rows of links m is decided, which gives the link length, $l = \beta H / m$, and width,

316 $b = l / \alpha$. By neglecting the slit width, the number of links in the direction of the wall's width is
317 obtained as $n = B / b$. The initial design is now complete.

318 Next, the width-to-thickness ratio of the link (b/t) should be checked considering the
319 sequence of nonlinear behavior. To dissipate more energy, individual links should undergo
320 sufficient in-plane plastification prior to the local buckling of links, which can be achieved by
321 having links with a small enough b/t . The b/t threshold that ensures in-plane behavior should be
322 determined by the specific steel. For the LYP100 in this study, the b/t threshold is proposed to be
323 five. If b/t is larger than the threshold after the initial design, a smaller b/t can be obtained by
324 increasing either α , t , or m . Several iterations may be needed before completion of the design
325 until b/t is below the threshold value.

326 **Conclusions**

327 The authors presented the design of SSSWs made from LYP100, which eliminated fracture
328 at link ends and showed a sound energy dissipation capacity without the need for out-of-plane
329 constraints. The major findings are summarized as follows:

330 (1) The feasibility of having the same strength and stiffness by adjusting link dimensions and
331 the number of rows of links was verified experimentally. This demonstrates the flexibility of the
332 SSSW design. The predicted yield strength and stiffness obtained using equations in previous
333 research agree well with experimental results.

334 (2) For the LYP100 steel, an upper boundary width-to-thickness ratio of five ensured the in-
335 plane behavior of individual links. By increasing the number of rows of links with the same
336 shear strength and stiffness, a small width-to-thickness ratio of the link was realized and “fat”
337 hysteretic loops were obtained without the need for out-of-plane constraints.

338 (3) The large strain hardening of LYP100 adjusted the strength of each row, resulted in
339 evenly deformed links in multiple rows, and dissipated more energy by the sufficient
340 development of plasticity in all links. Fracture was eliminated because of its large ductility.

341 (4) An SSSW design procedure was proposed, in which the aspect ratio and width-to-
342 thickness ratio of the link and the slit fraction are considered to meet the required shear strength
343 and stiffness while ensuring a good energy dissipation capacity.

344 (5) The combined hardening model in ABAQUS presented well the large strain hardening of
345 LYP100 and can be used to estimate the maximum strength and hysteric behavior of SSSWs
346 made from LYP100.

347 (6) Further work is required to make the proposed system suitable for practical applications.
348 The threshold value b/t that ensures stable hysteresis must be quantified. The design yield
349 strength of LYP, which is characterized by conspicuous strain hardening, must be determined.
350 Such strength is needed to estimate the energy dissipation of the slitted wall and the design of the
351 surrounding frame to ensure that it can sustain the strength transferred from the wall.

352 **References**

353 American Institute of Steel Construction (AISC) (2010). "Seismic Provisions for Structural Steel
354 Buildings." *ANSI/AISC 341-10*, Chicago.

355 Berman J. and Bruneau M. (2005). "Experimental investigation of light-gauge steel plate shear
356 walls." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(2005)131:2(259), 259-267.

357 Canadian Standard Association (CSA) (2009). "Design of steel structures." *CSA S16-09*,
358 Mississauga, ON, Canada.

359 Chen S. and Jhang C. (2006). "Cyclic behavior of low-yield point steel shear walls." *Thin-*
360 *Walled Structures*, 44(7), 730-738.

361 Chopra A. K. (2000). *Dynamics of Structures: Theory and Applications to Earthquake*
362 *Engineering* (2nd edition). Prentice Hall: New Jersey, USA.

363 Cortes G. and Liu J. (2011). “Experimental evaluation of steel slit panel frames for seismic
364 resistance.” *Journal of constructional steel research*, 67(2), 181-191.

365 Dassault Systèmes (2004). “ABAQUS Ver. 6.10 User’s Manual.” (<http://www.abaqus.com>)

366 Hitaka T., Ito M., Murata Y., and Nakashima M. (2009). “Seismic behavior of steel shear plates
367 stiffened by wood panels.” *Proc., Behavior of Steel Structures in Seismic Areas (STESSA)*,
368 Philadelphia, PA, 623-628.

369 Hitaka T. and Matsui C. (2003). “Experimental study on steel shear wall with slits.” *J. Struct.*
370 *Eng.*, 10.1061/(ASCE)0733-9445(2003)129:5(586), 586-595.

371 Hitaka T., Matsui C. and Sakai J. (2007). “Cyclic tests on steel and concrete-filled tube frames
372 with slit walls.” *Earthquake Engng Struc. Dyn.*, 36(6), 707-727.

373 Ke K. and Chen Y.Y. (2012). “Design method of steel plate shear wall with slits considering
374 energy dissipation.” *Proc., 15th World Conference on Earthquake Engineering*, Lisboa,
375 Portugal.

376 Ma X, Borchers E, Peña A, Krawinkler H, Billington S, Deierlein G. (2010). “Design and
377 behavior of steel shear plates with openings as energy-dissipating fuses.” John A. Blume
378 Earthquake Engineering Center Technical Report 173, Stanford Digital Repository.

379 Martinez-Rueda J. E. (2002). “On the evolution of energy dissipation devices for seismic design.”
380 *Earthquake Spectra*, 18(2), 309-346.

381 Matteis G., Landolfo R. and Mazzolani F. (2003). “Seismic response of MR steel frames with
382 low-yield steel shear panels.” *Engineering Structures*, 25(2), 155-168.

383 McCloskey DM. (2006). "Steel slit panels for lateral resistance of steel frame buildings." M.S.
384 thesis, West Lafayette (Indiana), Purdue University.

385 Nakashima M. (1995). "Strain-hardening behavior of shear panels made of low-yield steel. I:
386 test." *J. Struct. Eng.*, 10.1061/(ASCE)0733-9445(1995)121:12(1742), 1742-1749.

387 Nakashima M., Akazawa T., Tsuji B. (1995). "Strain-hardening behaviour of shear panels made
388 of low yield steel, II: Model." *J. Struct. Eng.*, 10.1061/(ASCE)0733-
389 9445(1995)121:12(1750), 1750-1757.

390 Qu B, Bruneau M, Lin CH, Tsai KC (2008). "Testing of full-scale two-story steel plate shear
391 wall with reduced beam section connections and composite floors." *J. Struct. Eng.*,
392 10.1061/(ASCE)0733-9445(2008)134:3(364), 364-373.

393 Roberts T. M. and Ghomi S. Sabouri (1991). "Hysteretic characteristics of unstiffened plate
394 shear panels." *Thin-Walled Struct.*, 12(2), 145-162.

395 Timoshenko S.P. and Gere J.M. (1961). *Theory of Elastic Stability* (2nd edition). McGraw-Hill:
396 New York.

397 Vian D. (2005). "Steel plate shear walls for seismic design and retrofit of building structures."
398 Ph.D. dissertation, SUNY at Buffalo: Buffalo, New York.

399 Zhang C., Zhang Z. and Shi J. (2012). "Development of high deformation capacity low yield
400 strength steel shear panel damper." *Journal of Constructional Steel Research*, 75, 116-130.

| | |
|-----|--|
| 401 | List of tables |
| 402 | Table 1. Summary of specimens |
| 403 | Table 2. Strength and stiffness |
| 404 | List of figures |
| 405 | Fig. 1. Steel slit shear wall |
| 406 | Fig. 2. Stress-strain relationship of LYP100 |
| 407 | Fig. 3. A slit shear wall tested: (a) test specimen; (b) hysteretic curves |
| 408 | Fig. 4. Details of specimens (unit: mm): (a)–(d) Specimens 1–4 |
| 409 | Fig. 5. Setup and instrumentation: (a) loading frame; (b) test setup; (c) placements of |
| 410 | displacement transducers and strain gauges (Specimen 1 for reference); (d) strain gauges in detail |
| 411 | Fig. 6. Incremental two-cycle loading |
| 412 | Fig. 7. Distribution of shear deformation among rows: (a)–(d) Specimens 1–4 |
| 413 | Fig. 8. Link buckling and fracture: (a)–(d) Specimens 1–4 |
| 414 | Fig. 9. Hysteretic curves: (a)–(d) Specimens 1–4 |
| 415 | Fig. 10. Equivalent damping ratios |
| 416 | Fig. 11. Energy dissipation |
| 417 | Fig. 12. Flow chart of design procedure |

418

Table 1. Summary of specimens

| Specimen | Material | t (mm) | b (mm) | l (mm) | m | α | β | b/t | Q_y (kN) | K (kN/mm) | Q_y / K (%) |
|----------|----------|----------|----------|----------|-----|----------|---------|-------|------------|-------------|---------------|
| 1 | LYP100 | 9 | 70.3 | 410 | 2 | 5.8 | 0.71 | 7.8 | 90.3 | 69.6 | 0.11 |
| 2 | LYP100 | 9 | 45.5 | 267 | 3 | 5.9 | 0.70 | 5.1 | 89.3 | 70.3 | 0.11 |
| 3 | LYP100 | 9 | 22.5 | 133 | 6 | 5.9 | 0.69 | 2.5 | 87.7 | 69.0 | 0.11 |
| 4 | SS400 | 4.2 | 32.4 | 243 | 2 | 7.5 | 0.42 | 7.7 | 103.0 | 36.0 | 0.25 |

419

Table 2. Strength and stiffness

| Specimen | $Q_{y,t} / Q_y$ | $Q_{1\%,t}$ (kN) | Q_p (kN) | $Q_{\max,t} / Q_p$ | K_t (kN/mm) | K_t / K |
|----------|-----------------|------------------|------------|--------------------|---------------|-----------|
| 1 | 0.99 | 181.2 | 135.4 | 2.09 | 68.5 | 0.98 |
| 2 | 1.04 | 182.9 | 134.0 | 2.38 | 69.9 | 1.00 |
| 3 | 1.16 | 187.5 | 131.5 | 2.24 | 68.2 | 0.99 |
| 4 | 1.06 | 180.6 | 154.4 | 1.25 | 35.8 | 0.99 |

420

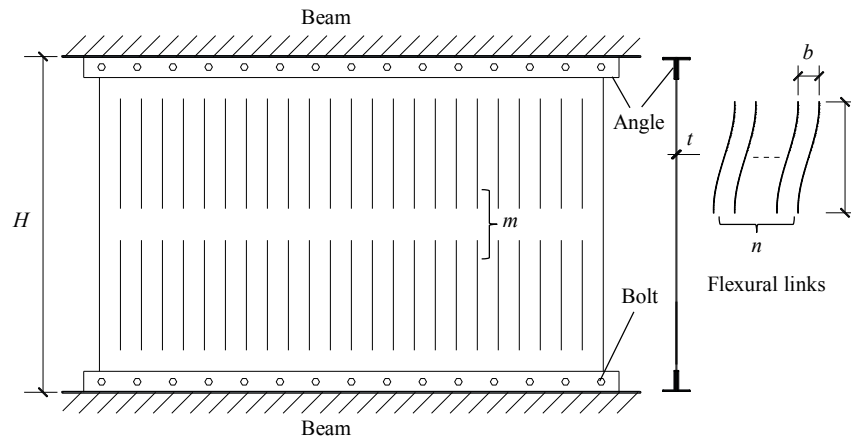


Fig. 1. Steel slit shear wall

421

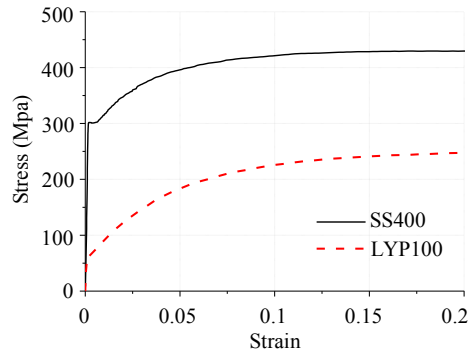


Fig. 2. Stress-strain relationship of LYP100

422

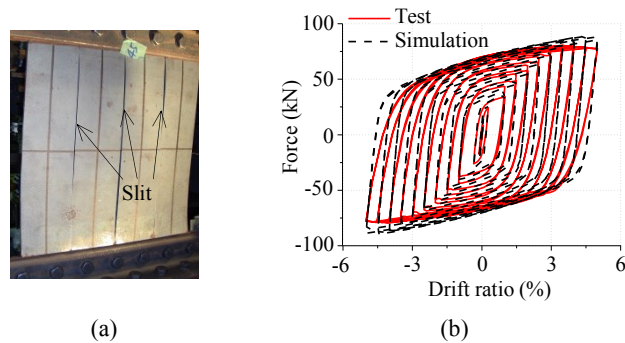


Fig. 3. A slit shear wall tested: (a) test specimen; (b) hysteretic curves

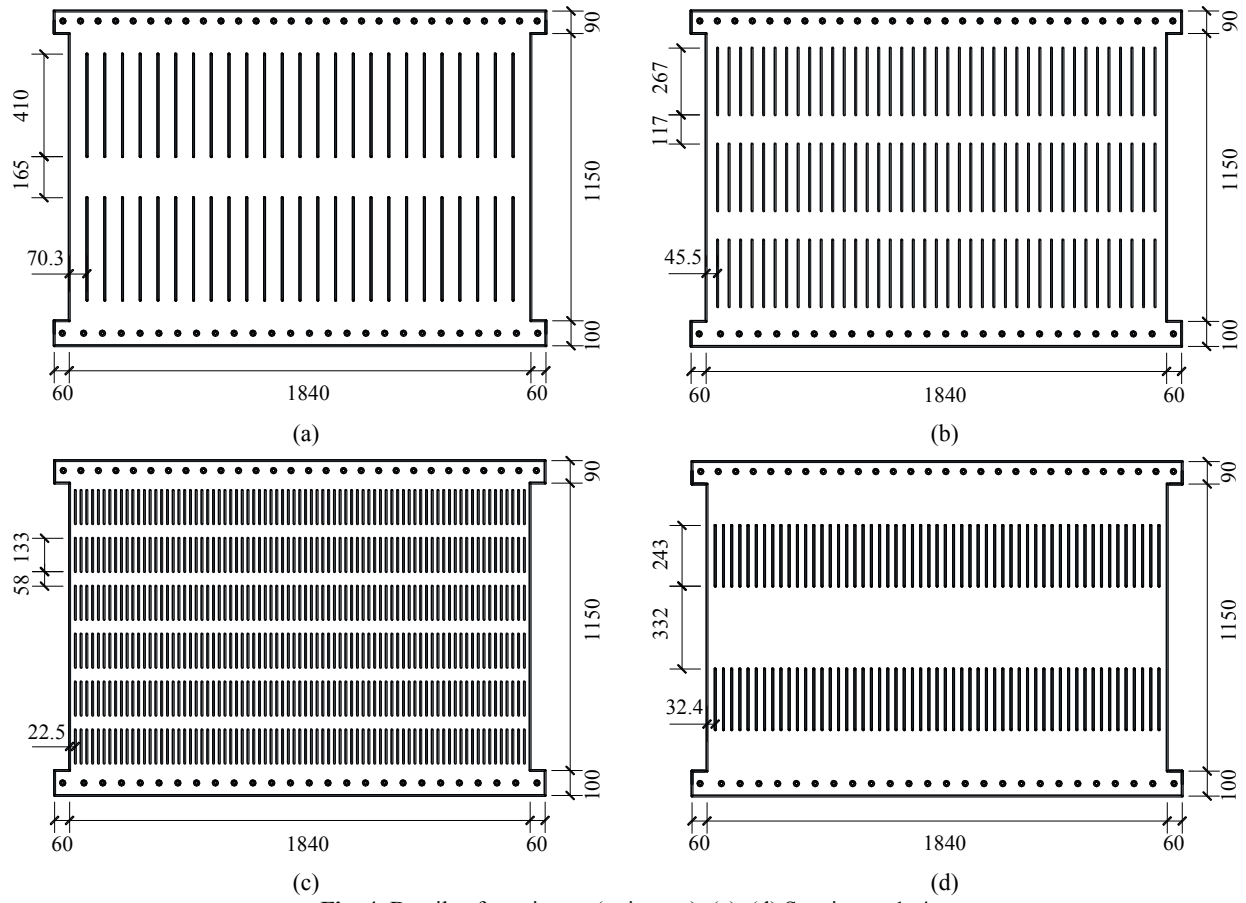


Fig. 4. Details of specimens (unit: mm): (a)–(d) Specimens 1–4

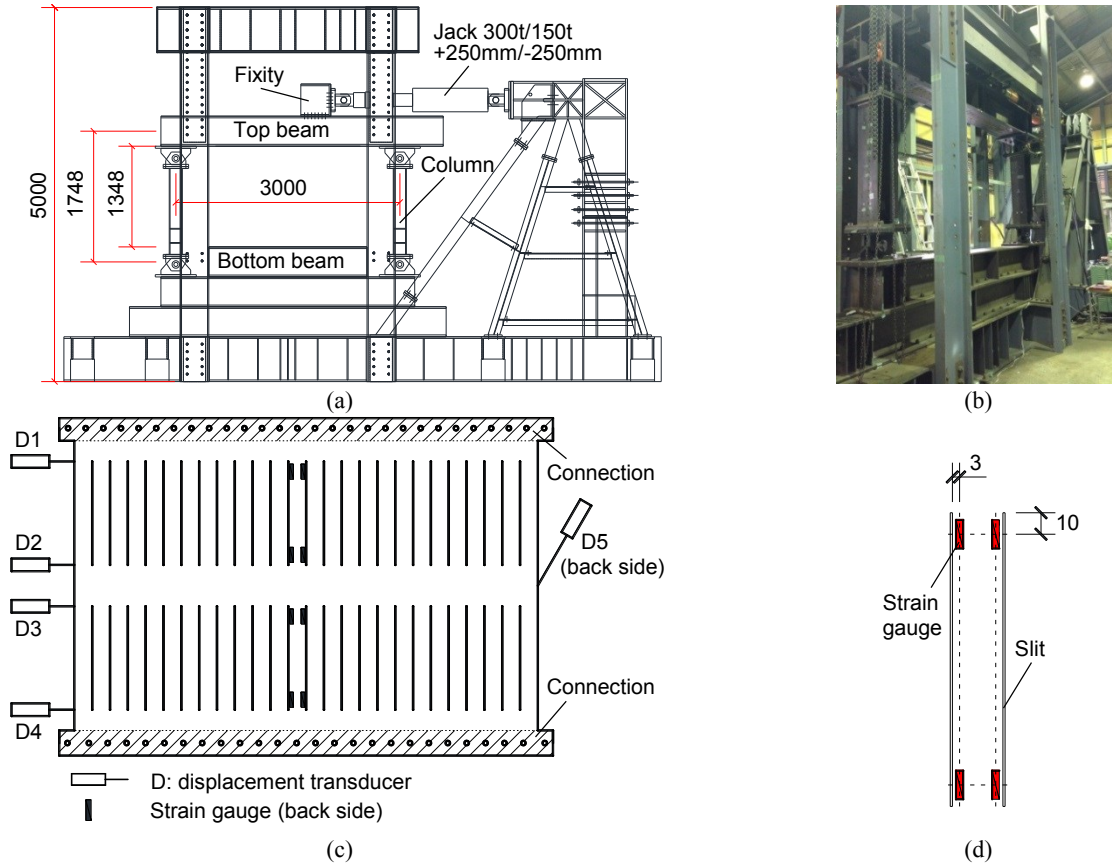


Fig. 5. Setup and instrumentation: (a) loading frame; (b) test setup; (c) placements of displacement transducers and strain gauges (Specimen 1 for reference); (d) strain gauges in detail

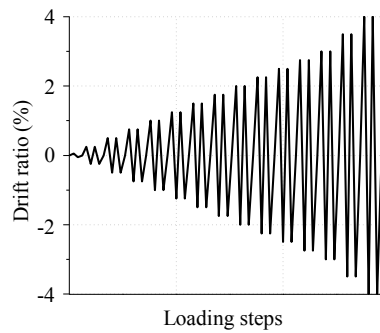


Fig. 6. Incremental two-cycle loading

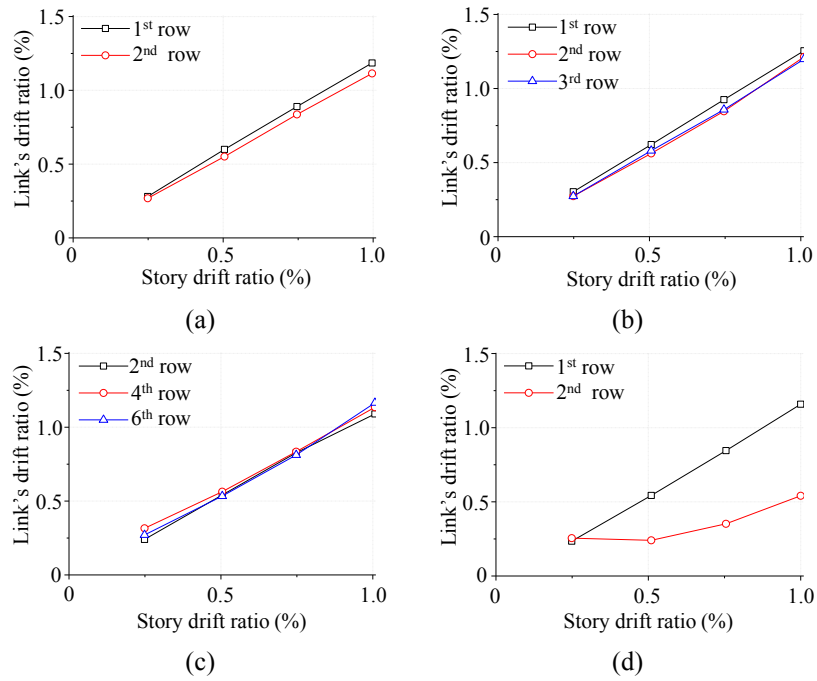


Fig. 7. Distribution of shear deformation among rows: (a)–(d) Specimens 1–4

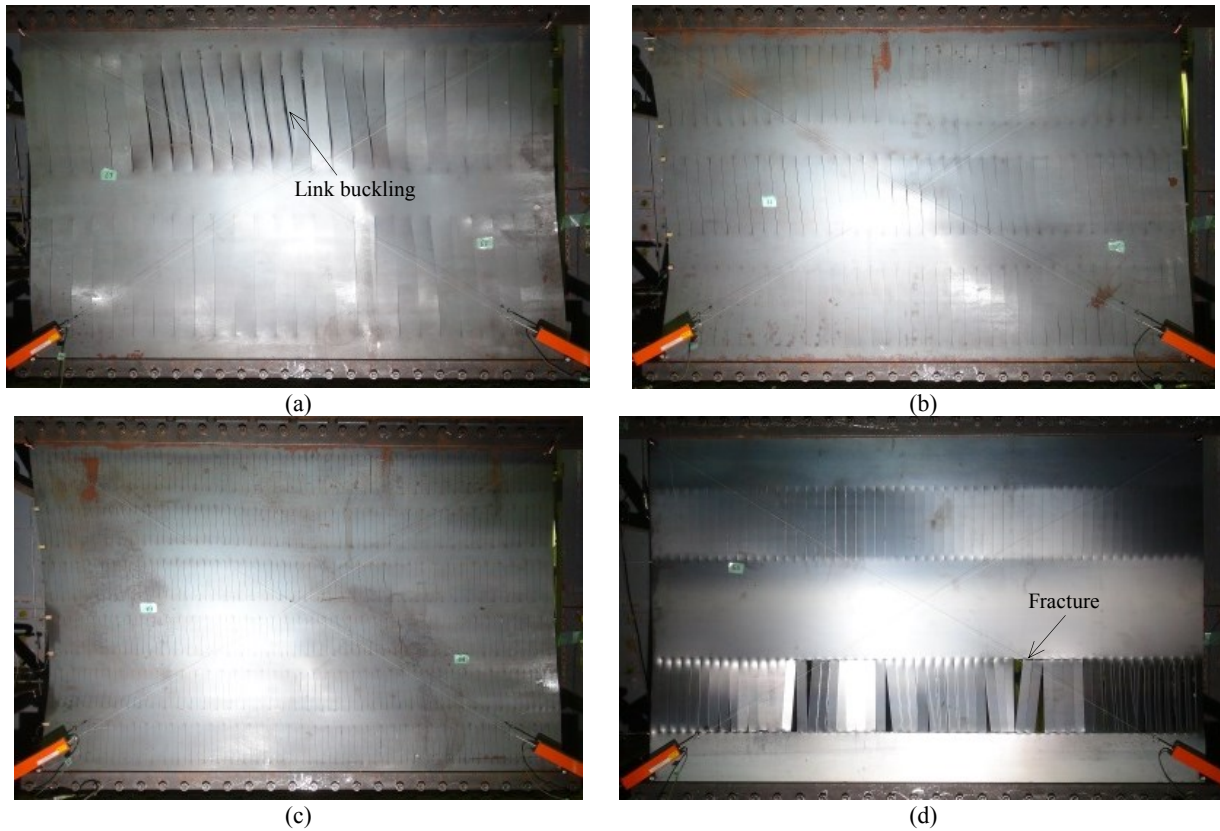


Fig. 8. Link buckling and fracture: (a)–(d) Specimens 1–4

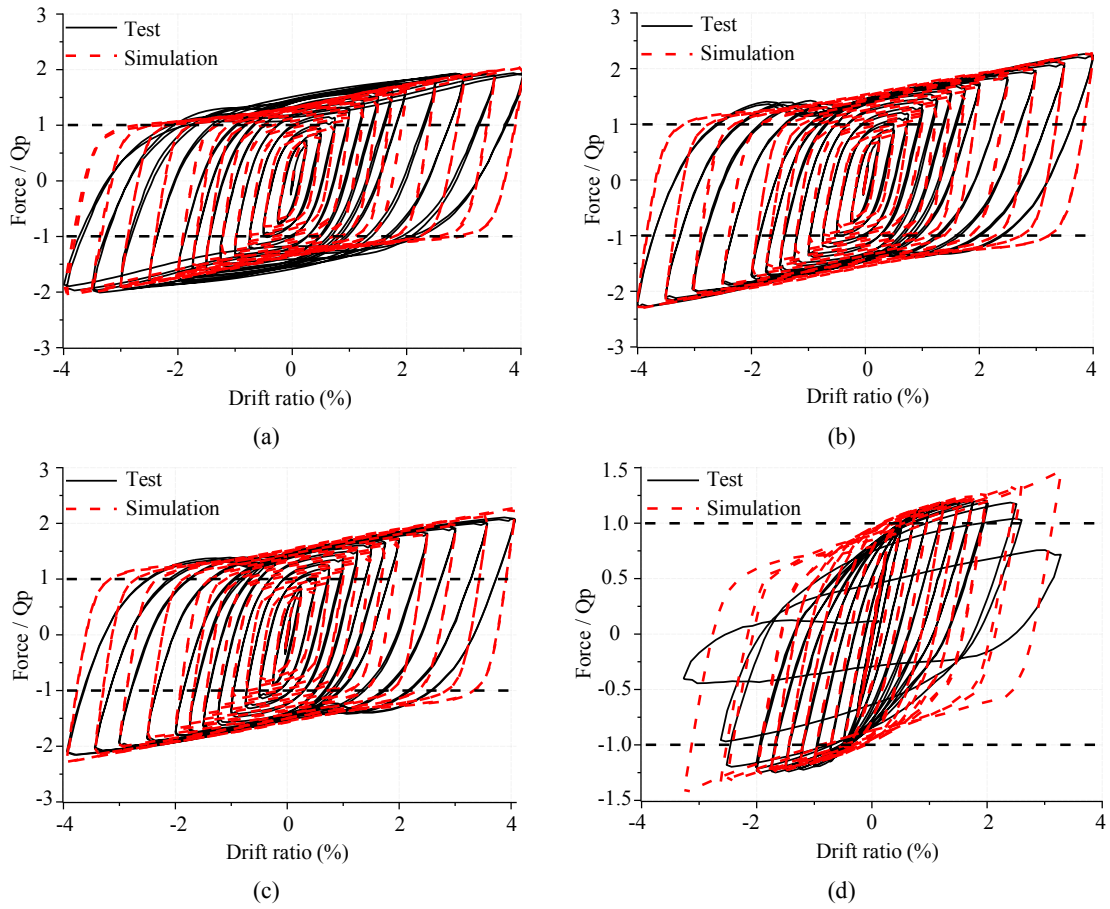


Fig. 9. Hysteretic curves: (a)–(d) Specimens 1–4

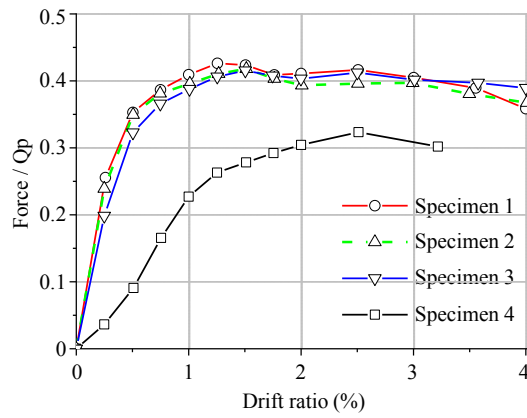


Fig. 10. Equivalent damping ratios

430

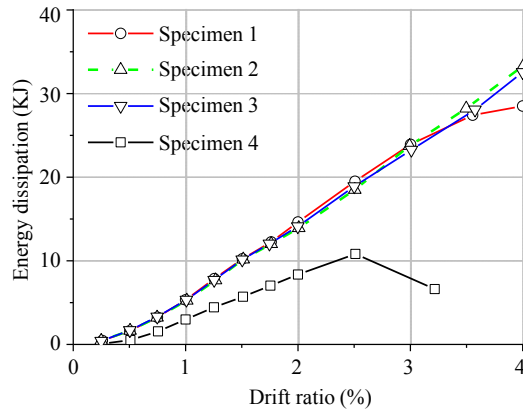


Fig. 11. Energy dissipation

431

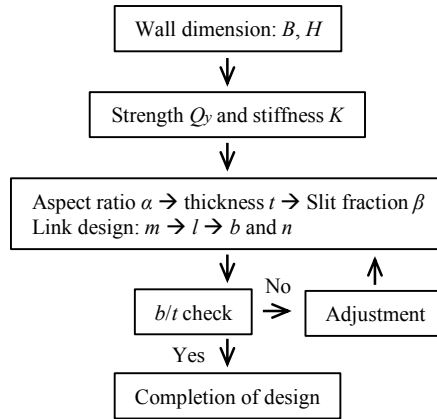


Fig. 12. Flow chart of design procedure