Seismic performance of subassemblies with composite wall and replaceable steel coupling beam

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1. Introduction

Coupled shear wall structures are highly efficient lateral load resisting systems. The ideal energy dissipation mechanism of such a system is illustrated in Figure 1(a) for a case with two coupled walls. Flexural plastic hinges should form at the end of all the coupling beams before the formation of plastic hinges at the base of wall piers. The shear forces, $V_c$, generated at the ends of the coupling beams are accumulated as a tension force, $T$, in one wall pier and a compression force, $N$, in the other pier (Figure 1(b)). These two forces are equal in magnitude and opposite in direction because the inflection points are assumed to be at the middle span of the coupling beams. The overturning moment produced by the horizontal load is partially resisted by $T$-$C$ couples and the ratio of their contribution to the total overturning moment is defined as the coupling ratio, denoted as $CR$. For a two-wall system, the $CR$ can be calculated using the following equation:

$$ CR = \frac{I \sum V_c}{I \sum V_c + I \sum M_i} \quad (1) $$

$$ \sum V_c = T = N \quad (2) $$

- $I$ is the lever arm between the centroids of two wall piers;
- $\Sigma V_c$ is the accumulation of vertical shear forces of the coupling beams;
- $M_i$ is the overturning moment resisted by wall $i$ ($i = 1$ or 2);
- $T$ is the tension force at the bottom of one wall pier;
- $N$ is the compression force at the bottom of the other wall pier.

For traditional reinforced concrete (RC) coupled shear wall systems, flexural plastic hinges at the ends of all coupling beams would be preferable to ensure satisfactory ductility and energy dissipation capacity. However, due to the architectural restrictions or the need for relatively high coupling ratio, coupling beams are often designed with a small span/depth ratio ($<2$). Shear failure would be inevitable for such cases, which will then result in poor seismic performance. Diagonally placed reinforcement is recommended (Park and Paulay 1975) to improve the seismic performance of RC coupled wall structures with short coupling beams. However, the connection between the diagonal bars and wall boundary element reinforcement poses construction challenges. Different type of newly
coupling beams, such as steel-concrete composite coupling beams (Gong and Shahrooz 2001), double skin composite coupling beams (Tae-Sung et al. 2009), embedded steel plate composite coupling beams (Wei et al. 2018; Hou et al. 2019) and replaceable steel truss coupling beam (Li and Liu 2019; Li et al. 2018), were also used to achieve better performance of the structures, yet the post-earthquake damage remained difficult to repair.

Well-detailed steel coupling beams with small span/depth ratio are stiff and strong, and they possess stable hysteretic behavior according to a number of past studies (Shahrooz Bahram, Remmetter Mark, and Fei 1993; Harries Kent et al. 1993; Park and Yun 2005; Cheng, Fikri, and Chen 2015; Bengar and Aski 2016; Li et al. 2018; Lu et al. 2018). In addition to achieving better performance, steel coupling beams offer economic advantages over RC coupling beams. Provisions for design and detailing of steel coupling beams have been developed and incorporated in the AISC seismic provisions (AISC 2016) based on the research conducted over the last two decades (Fortney et al. 2007a; Harries Kent et al. 1993; Harries et al. 1997; Shahrooz Bahram, Remmetter Mark, and Fei 1993, 2018; Qiang and Kurama Yahya 2002). To achieve post-earthquake reparability, “fuses” were introduced into steel coupling beams and studied both experimentally and analytically (Fortney et al. 2007b; Ji et al. 2017; Liu, Guo, and Shahrooz 2017; Shahrooz Bahram, Fortney Patrick, and Harries Kent 2018). This design concept is based on concentrating the damage caused by seismic loads in especially-detailed components, which can be removed and replaced after earthquakes allowing in situ repair of the structure. Steel coupling beams are expected to dissipate energy in a manner similar to shear links in eccentrically braced frames (EBFs) (Stratan and Dubina 2004; Mansour, Christopoulos, and Tremblay 2011; Azad and Topkaya, 2017). The idea of a mid-span fuse is analogous to a replaceable link in EBFs (Fortney et al. 2007b). The basic premise for coupling beams with a mid-span fuse coupling is that all inelastic deformations are concentrated in the fuse.

In super high-rise buildings, those exceeding 300 m in height, it is often difficult to resist the large axial forces and bending moments encountered in the lower stories of RC coupled walls. Composite shear walls are more advantageous for such cases. These composite alternatives include (a) steel reinforced concrete (SRC) shear walls (Xiaodong et al. 2017), (b) steel plate-reinforced concrete (SPRC) shear walls (Hu et al. 2016), and (c) concrete filled double-steel-plate composite shear walls (Nie et al. 2013; Hu, Nie, and Eatherton 2014; Varma et al. 2014).

Steel coupling beams in conjunction with SPRC shear wall piers would be a viable lateral load resisting system for super high-rise buildings. However, fundamental research into behavior of such system was limited. A new hybrid coupled wall structure consisting of replaceable steel coupling beams and SPRC shear wall piers was developed and studied experimentally (Figure 2). The main objective of the research presented in this paper was to further enhance the performance of steel coupling beams by providing post-event reparability without the costly repair of wall-pier-coupling beam connections. This objective was achieved by designing the beam such that all of the inelastic deformations would be concentrated in the middle of the steel coupling beam, i.e., where the fuse is located. The goal was to restore the resistance of the

Figure 1. Ideal energy dissipation mechanism of coupled walls with different coupling ratio.

Figure 2. Ideal energy dissipation mechanism of coupled walls with different coupling ratio.
structures after a seismic event. This paper focuses on the experimental aspect of the reported research.

2. Experimental program

2.1. Material properties

All the specimens were cast at the same time using concrete from a single ready-mix truck. The average 28-day compressive strength was 45.7 MPa, as determined by testing cubes. The maximum aggregate size was 10 mm, selected to ensure good consolidation around the embedded components. The horizontal reinforcement of wall piers and stirrups in the loading beam and pedestal consisted of 6-mm diameter deformed bars. The diameter of the longitudinal reinforcement in wall piers was 14 mm. The thickness of the steel plate and stiffeners was 6 mm.

The measured material properties are shown in Table 1.

2.2. Test specimens

Five specimens were fabricated and tested, with the parameters listed in Table 2. The specimen without a replaceable fuse was tested as benchmark. The reinforcement details of all the specimens were the same, as shown in Figure 3. The test variables were the connection details between the fuse and the rest of the coupling beam that was detailed to remain elastic, see Figure 4.

The length of all the coupling beam and fuse was 500mm and 250mm, respectively. Details “A” and “B” had the same stiffening but different connection details, as shown in Figure 4. Detail “A” is less effective in terms of moment and shear transfer compared with Detail “B”, but Detail “A” offers architectural and functionality advantages. The SPRC wall piers consisted of I-shaped steel boundary elements welded to “shear plates” and encased RC wall panels. The steel boundary elements were H125 × 125 × 6 × 9.

The elastic segments of the steel coupling beams were embedded in the wall piers and welded to the I-shaped steel boundary elements. As seen from Table 2, the thickness of flanges, webs, and end plates in the elastic components were made thicker than those in the fuse in order to prevent damage in the portions that are not meant to be replaced.

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Table 1. Material properties.

| Type        | d (mm) | fy (MPa) | fu (MPa) | Es (GPa) | εy (×10⁻⁶) |
|-------------|--------|----------|----------|----------|------------|
| Reinforcement | 6      | 625      | 630      | 2.0 × 10⁵ | 3125       |
|             | 14     | 455      | 595      | 2.0 × 10⁵ | 2275       |
| Steel       | 6      | 295      | 595      | 2.0 × 10⁵ | 1475       |

* d is the diameter of the steel bar or the thickness of the steel plate; fy is the yield strength; fu is the ultimate strength; Es is the elastic modulus; εy is the yield strain.

Table 2. Parameters of specimens.

| Specimen I.D. | Fuse or coupling beam | Elastic component | Span/depth | Replacement detail |
|--------------|-----------------------|-------------------|------------|-------------------|
| SCB-2        | H250 × 125 × 6 × 9    | –                 | 2.0        | –                 |
| FCB-1        | H250 × 125 × 6 × 9    | H250 × 125 × 12 × 20 | 2.0        | A                 |
| FCB-1-1*     | H250 × 125 × 6 × 9    | H250 × 125 × 12 × 20 | 2.0        | A                 |
| FCB-2        | H250 × 125 × 6 × 9    | H250 × 125 × 12 × 20 | 2.0        | B                 |
| FCB-2-1*     | H250 × 125 × 6 × 9    | H250 × 125 × 12 × 20 | 2.0        | B                 |

*with a new fuse
High strength 16-mm diameter bolts were used to connect the end plates of the “fuse” and the elastic components of the coupling beam. The ultimate strength of the bolts (designated as Grade 10.9) was 1000 MPa with the yield strength to ultimate strength ratio of 0.9. Shear stud connectors were used on both sides of the steel plates and the flanges of the boundary elements to ensure sufficient bond between the steel and concrete. The diameter and nominal length of the studs were 16 mm and 60 mm, respectively.
The shear capacity of each slip-critical high strength bolt can be calculated using Eq. (3) based on Chinese code (GB-50017 2017):

\[ N_s = 0.9n/P \mu \]  

(3)

- \( n \) is number of contact surfaces;
- \( \mu \) is friction coefficient, which is 0.45 for sanded Q235 steel;
- \( P \) is prestressed axial load of the bolts, which is 100kN for 16-mm Grade 10.9 high strength bolts.

The shear capacity of each bearing type bolt can be calculated using Eq. (4):

\[ N_s = n_s \pi d f_s / 4 \]  

(4)

- \( n_s \) is number of shear planes,
- \( d \) is bolt diameter,
- \( f_s \) is design shear strength taken as 310 MPa for Grade 10.9 bolts.

The designed shear capacity of the high-strength bolt groups, both for slip-critical and bearing type, are listed in Table 3. Due to limited space of the end plates in Detail “A”, only six bolts could be installed in each end plate to meet the specification of minimum distance center to center of bolts, which is insufficient compared with the shear demand corresponding to the fully developed flexural capacity of the steel beam.

### 2.3. Experimental setup

The test setup is shown in Figure 5. The specimens consisted of three parts: steel coupling beam, two SPRC wall piers, and two transfer beams. For ease of loading, the specimens were rotated 90 degrees such that the coupling beam was vertically placed (Figure 2(c)), with the bottom transfer beam being fixed on the pedestal and the upper transfer beam being bolted to a rigid L-shaped loading girder. Three actuators were used to simulate the loading and boundary conditions expected in coupling beams of coupled wall buildings (Figure 2(b)). The specimen was loaded horizontally with a 1000-kN hydraulic actuator to create a lateral chord rotation between the ends of the coupling beam. Two 500-kN hydraulic vertical actuators were attached to the steel L-shaped loading girder and lab’s strong floor, forming a parallelogram system. The parallelogram system can limit rotation of the loading girder, thereby reducing rotation of the wall pier. Two screw jacks were arranged on both sides of the bottom transverse beam of the specimen to limit its horizontal displacement. To guarantee lateral stability of the loading system, the L-shaped loading girder was laterally braced at three points.

All the specimens were loaded similarly following a series of cyclic displacements with increasing amplitudes. The selected loading protocol is shown in Figure 6. Specimen SCB-2 was loaded up to 10% chord rotation. Specimens FCB-1 and FCB-2 were first loaded up to 2% chord rotation. After conducting 3 cycles at this displacement amplitude, loading was stopped with zero horizontal load. The small residual chord rotation and termination of loading simulated the state of the structure after experiencing a moderate earthquake (Shahrooz Bahram, Fortney Patrick, and Harries Kent 2018). In-situ replacement of the fuse is shown in Figure 5(a). The residual displacement in FCB-1 or FCB-2 with zero lateral load was maintained with the three actuators described above. After installing new fuses, specimens FCB-1-1 and FCB-2-1 were loaded up to 10% chord rotation, with the ending state of the loading history prior to replacing the fuse being taken as the starting point. It has to be mentioned that three cycles at 10% chord rotation could not be completed for specimen FCB-1-1 mainly due to instability issues that will be explained later in the paper.

### 2.4. Instrumentation and measurements

Instrumentations, including load cells, linear variable differential transformers (LVDTs) and strain gauges, were used to measure load, displacement, and strains in the specimens, respectively, as shown in Figure 7. Two non-contact laser displacement transducers (LDTs) were used to measure the horizontal displacement of the wall pier edge adjacent to the end of the coupling beam. The difference between these two displacements divided by the clear span of the coupling beam is the chord rotation, denoted as ϑ. Two pairs of LVDTs were diagonally attached on both sides of the steel coupling beam to measure and then calculate the local and global shear deformations of the beam. Another LVDT was used to monitor the possible slippage of the transfer beam fixed on the pedestal. Strain gauges and rosettes were mounted to the surface of embedded steel plates, boundary elements, and steel coupling beams. Their arrangement is shown in Figure 7(b).

### 3. Experimental results

#### 3.1. Damage and crack pattern

Typical failure modes are shown in Figure 8. For specimen SCB-2, some minor cracks on the wall piers were observed at small chord rotation (around 0.3%). The flanges of the steel coupling beam yielded during the

| No. | \( M_s / \text{kNm} \) | \( V_s / \text{kN} \) | \( \Sigma N^s_1/\Sigma V_0 \) | \( \Sigma N^s_2/\Sigma V_0 \) | \( \Sigma N^s_3/\Sigma V_0 \) |
|-----|------------------|------------------|------------------|------------------|------------------|
| FCB-1-1 | 91.75 | 367.0 | 243.0 | 0.66 | 324.0 | 0.88 |
| FCB-2-1 | 91.75 | 367.0 | 373.8 | 1.02 | 498.4 | 1.36 |

\( M_s \) is the calculated flexural strength of the controlled specimen SCB-2 and \( V_s \) is the corresponding shear force; \( \Sigma N^s_1 \) and \( \Sigma N^s_2 \) are the designed shear strength of each bolt group calculated under slip-critical and bearing situation, respectively.
first cycle of 1% chord rotation. The flanges of the steel beam began to separate from the wall piers at 2% chord rotation. Buckling of the flanges was observed at the first cycle of 5% chord rotation and became progressively severe during the following loading stages. Localized spalling of concrete at the edge of the top wall pier occurred at the first cycle of 10% chord rotation, as indicated by the upper insert in Figure 8(a). Local buckling of the web was observed at the last cycle of 10% chord rotation, and diagonal tension fields began to propagate at the bottom end of the steel coupling beam, which can be seen in the lower inset in Figure 8(a). A slight strength degradation was detected only during the last loading cycle, see Figure 9(a).

For specimens FCB-1, no discernible cracks were detected until 1% chord rotation. Some minor cracking occurred at the beam-wall interface, and the web of the fuse began to yield due to diagonal tension. At 2% chord rotation, there was a 1.5 mm gap between the end plates, and the maximum value of tensile strain in the fuse web was more than 2000 με. Plastic deformation had been developed in the fuse to some extent under this displacement amplitude, even though Detail “A” is less efficient in moment and shear transfer compared with Detail “B”.

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**Figure 5.** Test setup.
For small chord rotations, there was no major difference between specimens FCB-1-1 and FCB-1. As the displacement amplitudes increased, the gap between the two end plates and the plastic strain in the fuse web became larger. The developing gap (Figure 8(b)) and the reversed loading weakened the...
resistance of the coupling beam, as evident by several sudden drops of the horizontal load (Figure 9(b)) and loud noises due to slippage between the end plates. This phenomenon can be clearly demonstrated from the inserts in Figure 8(b) and the hysteretic curve shown in Figure 9(b). For safety reasons, specimen FCB-1-1 was monotonically loaded from 5% chord rotation up to 10% chord rotation, which was the maximum intended chord rotation. No weld fracture or strength degradation was observed. It should be noted again that Detail "A" used in FCB-1 is not expect to provide significant moment transfer, despite of its architectural and constructability benefits.

For specimens FCB-2, no major issues were observed except for a few minor cracks in the wall piers. The fuse web yielded at 1% chord rotation, which is earlier than specimen FCB-1 and FCB-1-1. For specimen FCB-2-1, no gap between the end plates was observed during the entire loading history (Figure 8(c)), mainly because of the connection details (Figure 4(b)) that could transfer moment and shear force between the fuse and the rest of the coupling beam more efficiently than FCB-1-1. Initiation of weld

Figure 9. Hysteretic curves of horizontal load vs. drift.
fracture in the flange stiffener was detected at 5% chord rotation. The fuse flanges buckled during the first cycle of 5% chord rotation, and the fuse web-stiffener weld began to fracture at the third cycle of 6.7% chord rotation. During the first half cycle of 10% chord rotation, there was a sudden drop in the horizontal load (Figure 9(d)) due to web fracture at the web-flange interface and propagation of fracture in the web-stiffener weld. Loading was stopped at the end of the first half cycle of 10% chord rotation. The final failure mode of specimen FCB-2-1 is shown in Figure 8(c).

3.2. Hysteretic response

The horizontal load (P)- chord rotation (θ) hysteretic curves of all the specimens are plotted in Figure 9. The curves of specimen SCB-2 are stable and show very good energy dissipation characteristics. Only slight strength degradation was detected for the second cycle of 10% chord rotation at which the web began to buckle. The hysteretic performance of specimen FCB-1-1 below 2% chord rotation is almost identical to that of FCB-1 (Figure 9(c)), suggesting that replacing fuses is feasible after moderate earthquakes. For specimen FCB-2, equipment failure negatively impacted the hysteretic performance. As a result, the responses of this specimen and FCB-2-1 are slightly different (Figure 9(e)). The results, nevertheless, demonstrate that the seismic performance of the specimen could be restored after replacing the damaged fuse. The effects of excessive opening of the end plates in FCB-1-1 is evident by a number of sudden load drops (Figure 9(b)). For specimen FCB-2-1, the connection between the fuse and the elastic components was modified by extending the ends plates above and below the flanges and using flange stiffeners. The hysteretic performance of FCB-2-1 was improved remarkably in comparison to specimen FCB-1-1 and was nearly similar to that of specimen SCB-2. As stated before, moment transfer in specimen FCB-1-1 is not expected to be large yet offers more architectural functionalities than FCB-2-1.

3.3. Skeleton curve

The skeleton curves of all the specimens are shown in Figure 10, and the characteristic points of the curves are summarized in Table 4. The nominal yielding point of the skeleton curve was determined graphically according to the well-established equivalent elastoplastic energy absorption procedures (Park 1989). For specimen FCB-2-1, the ultimate point at positive loading direction was chosen as the maximum displacement at 6.7% chord rotation because loading was stopped before reaching the next maximum chord rotation due to rupture of the fuse web. For the other specimens, the ultimate point of the skeleton curve coincides with the maximum strength point, since the skeleton curve of specimens maintained an upward trend during the entire loading process.

The ultimate capacity of specimen SCB-2, FCB-1-1, and FCB-2-1 was 330.3kN, 338.3kN, and 425.1kN in forward loading, respectively. The specimens after replacing the fuse exhibited nearly identical strength in comparison to their former counterpart specimens, which indicates the fuse can reasonably be replaced. The maximum strength of specimen FCB-1-1 and FCB-2-1 was 17.8% and 35.8% higher than that of SCB-2, respectively. The higher

Figure 10. Skeleton curves of specimens.
capacities are attributed to the smaller span-to-depth ratio of the fuses in FCB-1-1 and FCB-2-1.

### 3.4. Energy dissipation characteristics

Dissipated energy $E_d$ and equivalent viscous damping coefficient $\zeta_{eq}$ (JGJ/T 101 2015) were determined to evaluate the energy dissipation performance of specimens. The definition of these two indices were shown in Figure 11. For a given hysteretic loop, the contained area indicated as the shaded part in Figure 11 is the dissipated energy $E_d$. The equivalent viscous damping coefficient $\zeta_{eq}$ was calculated based on the following equation:

$$\zeta_{eq} = \frac{1}{2\pi} \frac{S_{OABCDEF}}{S_{OAG} + S_{ODH}}$$  \hspace{1cm} (4)

- $S_{OABCDEF}$ is the shaded area, which is the dissipated energy $E_d$;
- $S_{OAG}$ and $S_{ODH}$ are the areas of the two dashed triangles.

The dissipated energy $E_d$ and the equivalent viscous damping coefficient $\zeta_{eq}$ under different chord rotations are listed in Tables 5 and 6, respectively. The dissipated energy, expectedly, increased with an increase in the chord rotation. Specimens FCB-1-1 and FCB-2-1 dissipated more energy than SCB-2 up to 5% chord rotation. These dissipated energies suggest the adequacy of replacement details. As the chord rotation increased, the equivalent viscous damping coefficient, expectedly, increased, indicating that the plastic energy dissipation of the specimen became larger. As mentioned before, there were several sudden slippages of the end plates in specimen FCB-1-1 during the loading process. These slippages reduced the amount of dissipated energy; hence, the equivalent viscous damping coefficient of FCB-1-1 was slightly lower than the other specimens.

### 3.5. Stiffness characteristics

The degradation of peak-to-peak stiffness against chord rotation is plotted in Figure 12. The initial stiffness of all the specimens, expectedly, degraded with an increase in the chord rotation. No significant differences in the overall trend of the data are observed in general. The initial stiffness of specimen SCB-2 is lower than the other four specimens, because the thickness of flanges and web in this specimen were smaller than their counterparts in the specimens with a mid-span fuse.

### 3.6. Measured strains

#### 3.6.1. Coupling beam flange strain

The hysteretic curves of the horizontal load vs. flange strain (positive is tensile and negative is compressive)
for the steel coupling beams are shown in Figure 13. Flange strain at both the middle and end sections of specimen SCB-2 and FCB-1-1 were selected as shown in Figure 13(a)–(d). For specimen FCB-2-1, due to the arrangement of triangular stiffeners at the end of the coupling beam, there was not enough space to install strain gauges. Therefore, only flange strain in the middle section of the fuse for specimen FCB-2-1 is presented in Figure 13(e).

From Figure 13(a), it is observed that the flange strain at the end of specimen SCB-2 increased gradually as the chord rotation was amplified. The rate of increase became more pronounced after flange buckling. The maximum flange strain in the end section was well above the yield strain, suggesting formation of plastic hinges at the end of steel coupling beam. The strain in the middle section of specimen SCB-2 was much lower than that at the end section, as shown in Figure 13(b). This result is another evidence that specimen SCB-2 developed a flexural hinge mechanism, similar to that shown in Figure 1(a).

For specimens FCB-1-1 and FCB-2-1, the trend of the results is reversed. The maximum flange strain at the end section of specimen FCB-1-1, which is also the elastic part, is less than the yield strain, as shown in Figure 13(c). Figure 13(d)–(e) demonstrate that flanges of the “fuses” for specimens FCB-1-1 and FCB-2-1 had fully yielded both in tension and compression, mainly due to formation of tension fields in the beam web. Therefore, specimens FCB-1-1 and FCB-2-1 developed “shear hinge” mechanism as suggested in Figure 1(c).

### 3.6.2. Coupling beam web strain

Strain rosettes were used to measure and then calculate the principle tensile strain of the beam web. The hysteretic curves of the horizontal load vs. web strain for all three steel coupling beams are shown in Figure 14. It is seen from Figure 14(a) that the principle web strain of specimen FCB-2-1 increased rapidly as the horizontal load became large, suggesting formation of sufficient tension field in the web and further demonstration of the feasibility of replacement detail “B”. Meanwhile, the inefficiency of Detail “A” in transferring moment and shear is also clearly evident from the lower level of principle web strains shown in Figure 14(b). For specimen SCB-2, it can be seen from Figure 14(c) that the principle web strain reached the yield strain only at the end of the steel coupling beam and after web buckling.

**Table 5.** Dissipated energy (kJ) at different story drift angles.

| Specimen No. | (1/100)  | (1/75)  | (1/50)  | (1/35)  | (1/20)  | (1/15)  | (1/10)  | Dissipated energy before θ = 0.05 | Total dissipated energy |
|--------------|----------|----------|----------|----------|----------|----------|----------|----------------------------------|------------------------|
| SCB-2        | 1.89     | 4.35     | 10.93    | 21.44    | 54.88    | 86.27    | 98.79    | 93.50                            | 278.55                 |
| FCB-1        | 1.96     | 4.01     | 10.12    | **       | **       | **       | **       | 16.09                            | 16.09                  |
| FCB-1-1      | 1.38     | 3.49     | 9.90     | 20.45    | 51.81    | **       | 17.38*   | 87.03                            | 104.41                 |
| FCB-2        | 0.53     | 1.89     | 8.01     | **       | **       | **       | **       | 10.43                            | 10.43                  |
| FCB-2-1      | 1.34     | 3.97     | 11.33    | 23.67    | 66.05    | 104.79   | 16.78*   | 106.37                           | 227.95                 |

*Not finished cycle at this drift  **No cycle at this drift

**Table 6.** Equivalent viscous damping ratios at different drift angles.

| Specimen No. | (1/100)  | (1/75)  | (1/50)  | (1/35)  | (1/20)  | (1/15)  | (1/10)  |
|--------------|----------|----------|----------|----------|----------|----------|----------|
| SCB-2        | 0.16     | 0.22     | 0.29     | 0.35     | 0.43     | 0.46     | 0.50     |
| FCB-1        | 0.15     | 0.19     | 0.26     | **       | **       | **       | **       |
| FCB-1-1      | 0.11     | 0.17     | 0.25     | 0.31     | 0.39     | **       | **       |
| FCB-2        | 0.06     | 0.09     | 0.21     | **       | **       | **       | **       |
| FCB-2-1      | 0.09     | 0.17     | 0.27     | 0.33     | 0.43     | 0.46     | *        |

*Not finished cycle at this drift  **No cycle at this drift

**Figure 12.** Stiffness vs. drift curve.
3.6.3. Steel plate strain

Several equally-spaced strain rosettes were mounted to the SPRC wall plates and webs of the boundary elements, as shown in Figure 7(b). The principle tensile strain at the instrumented points under each positive displacement amplitude for specimen FCB-1-1 and FCB-2-1 was then calculated and shown in Figure 15. The principle tensile strain increased as the displacement amplitude became larger. The joint panel of the I-shaped boundary element and the steel coupling beam for specimen FCB-2-1 yielded at 2% chord rotation, suggesting that large moment was transferred into the wall pier from the end of coupling beam. This trend is consistent with the high efficiency of moment transfer expected for Detail B. The strain levels decreased rapidly with an increase in the distance away from the edge of wall pier. The steel plates in both wall piers remained in the elastic range.

Figure 13. Hysteretic curves of horizontal load vs. flange strain of steel coupling beams.
4. Conclusions

Subassemblies consisting of two SPRC wall piers and a steel coupling beam with a mid-span fuse were tested to evaluate the seismic performance of this system, and to examine the feasibility of replacing the fuse. A total of five experiments were conducted and the replacement procedure was demonstrated. The following conclusions can be drawn based on this research:

1. The specimens exhibited satisfactory performance, including: high strength and stiffness, excellent ductility, and stable hysteretic behavior. Based on
the experimental results, both of the selected fuse details performed well in terms of restoring the seismic performance after subjecting the specimen to 2% chord rotation and replacing the fuse. The initial stiffness of the specimens with fuses was higher than that with a single steel coupling beam, because of the thicker flanges and webs in the elastic components of the former specimens.

(2) The steel coupling beam with replacement detail “A” experienced some sudden slippages between the end plates under large rotation angles, which led to brief losses of load carrying capacity and degradation of energy dissipation capacity. It should be noted that the selected detail is not expected to offer a large level of moment transfer. This problem was successfully remedied using replacement detail “B”.

(3) No significant damage was observed in the wall piers, especially for the specimens with fuses. It is concluded that the selected replacement details offer an effective method to limit the entire plastic damage to within the fuse.

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Data availability statement

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