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Development of Innovative Lateral Resistance Systems Featuring Earthquake-Protective Dampers

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Abstract: Several conventional structural systems require sufficient retrofitting design procedures, improvements, and reconstructions to withstand lateral loads and to decrease the occurrence of damage. High strength capacity and ductility for seismic lateral resisting systems improve the structural vulnerabilities and limit damage concentrations in areas subject to seismic conditions. Several types and shapes of structural systems with appropriate ductility and energy dissipation features are currently established as structural fuses to enhance the general performance of the structures and decrease seismic ramifications. To enhance the energy dissipation performance and concentration of the inelasticity, improving the ductile behavior and limiting the unpredictable accumulation of plastic strains is essential. The conventional eccentrically braced systems are examined and reestablished, and the effects of shear fuses used in high-rise buildings are investigated for prototype buildings by implementing the verified simulations. Next, seismic protective fuse systems with innovative dampers consisting of several butterfly-shaped shear links are established. Ultimately, the design guidelines are established based on the conventional eccentrically braced frames (EBFs), which are redesigned with the use of noble seismic protective fuses, and the hysteretic behavior is obtained and compared accordingly.

Keywords: structural fuses; finite element analysis; computational programming; new generation of lateral resisting systems

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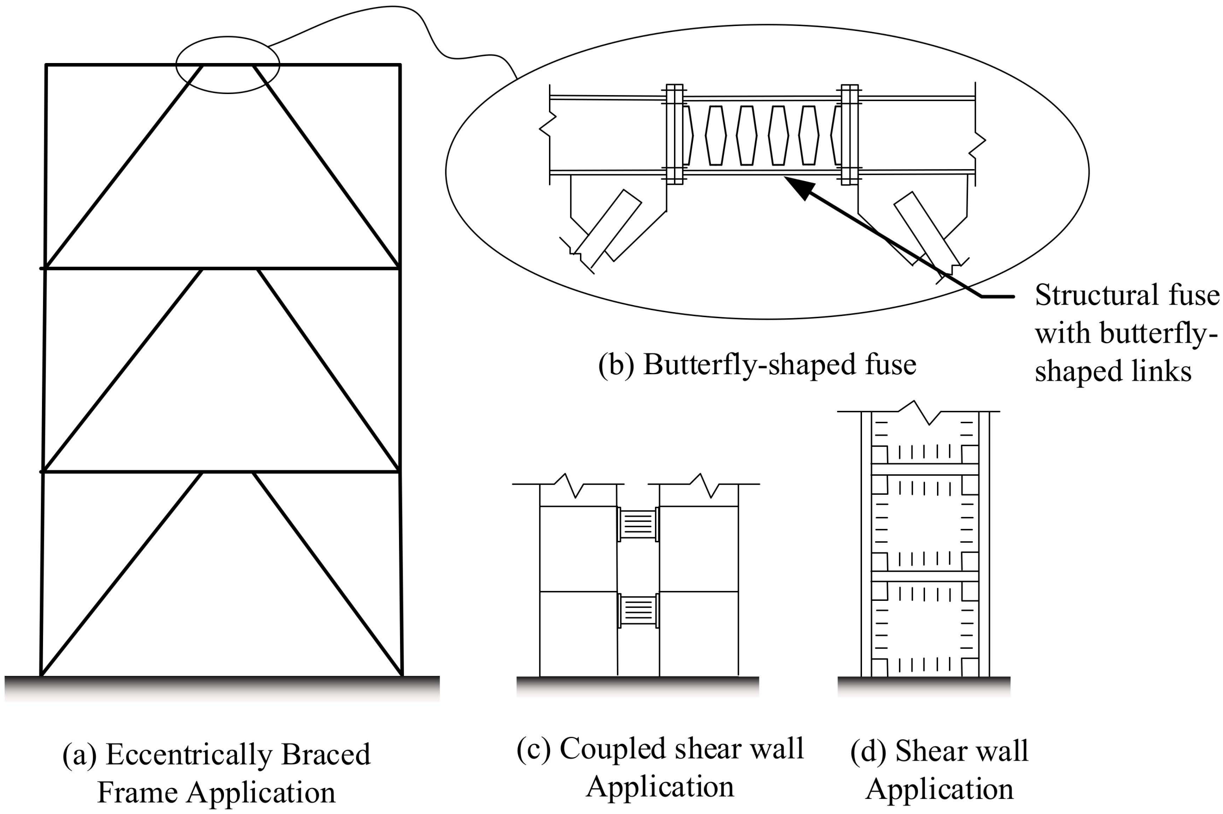
Citation: Farzampour, A.; Mansouri, I.; Mortazavi, S.J.; Retzepis, E.; Kaloop, M.R.; Hu, J.-W. Development of Innovative Lateral Resistance Systems Featuring Earthquake-Protective Dampers. *Appl. Sci.* **2023**, *13*, 3852. https://doi.org/10.3390/app13063852

Academic Editor: Gianfranco De Matteis

Received: 9 February 2023
Revised: 14 March 2023
Accepted: 15 March 2023
Published: 17 March 2023

1. Introduction

During major earthquakes, the ductile mechanism incorporated in structural applications provides the inelastic drift capacity to improve general energy dissipation and avoid significant damages [1–3]. For various applications, the shear fuse system could be handled through the strategic removal of the substance to uniformly concentrate inelasticity over the length, in order to save other elements from unpredicted damages and preserve their intactness [4–9]. A desirable structural fuse type recently used in different infrastructure is hourglass-shaped beams, for which the steel web consists of semi-hourglass shape cutouts, strategically leaving the rest of the steel for approximating the moment capacity diagrams with moment demand diagrams [10–13]. The geometric details of the hourglass-shaped beam implemented in mid-rise infrastructures and buildings are shown in Figure 1. Structures are created with the intention of enduring earthquakes without being destroyed. To minimize harm to the structural system, engineers use steel shear dampers that have the ability to undergo inelastic drift and dissipate energy when exposed to excessive loading conditions [14–17]. These dampers function as structural fuses, directing inelasticity to a specific part of the structure to protect other components from damage. The use of dampers is a common method used to mitigate the effects of seismic loading on structures, but their effectiveness in structures built with green concrete is not well understood [18]. Steel plates with shear links cut into them are an example of such dampers that possess adequate ductility and energy dissipation capability, enabling them to undergo yielding ductile modes of behavior when exposed to shear loading demand.



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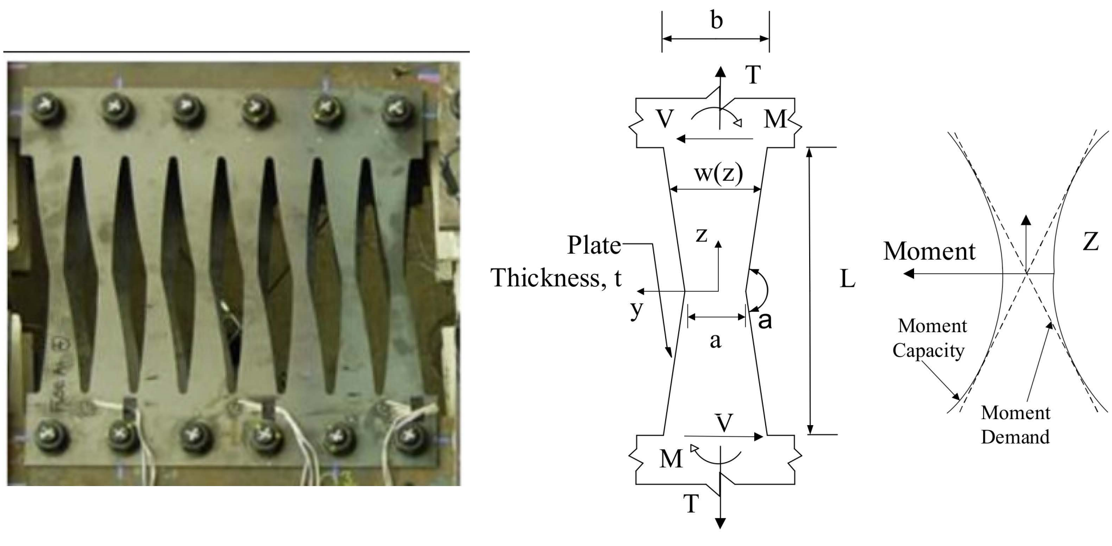


(**B**)

Figure 1. Butterfly-shaped link and its uses: (**A**) hourglass-shaped fuse systems in various applications; (**B**) shear fuse system loading conditions.

Various prototype dampers, including the butterfly-shaped web steel units, have recently been established for the better concentration of inelasticity and plasticity within a pre-determined region. Similar strategies to protect a beam column's interconnection area from significant damages are used for reestablishing various multistory buildings and for the fortification purposes of existing structures [13]. The use of fuses substantially increases the energy structural performance in highly seismic areas [6,12,19–21]. The drift response of buildings could be controlled by using structural fuses, as they would lead to more effective energy dissipation and lower the external load on the structural frame [22]. Butterfly-shaped fuses provide not only substantial energy dissipation but also ductility and a large, evenly distributed yield when used in-plane. This makes them a suitable choice for dampers in high-rise buildings, as they can limit the drift response and reduce demands on the framing. Previous design methods for dampers have been based on flexural limit states, which can result in an over- or under-estimation of the dampers' capacity against lateral forces. Studies have shown that stress is not evenly distributed along the length of the dampers, making it necessary to improve the design methodology and conduct effective investigations of steel dampers.

New structural dampers could accumulate the plastic strains within the structural shear fuses, whereas the rest of the boundary elements remain intact. Several works found that these dampers could experience shear angle ratios of more than 30% without crack initiation [23,24]. The general butterfly-shaped fuse system and the related loading condition are shown in Figures 1A and 1B, respectively.

Various studies have tackled the common challenges associated with structural fuse systems containing different types of dampers. It has been determined that butterfly-shaped dampers are effective because they align the demand moment curves with the capacity moment diagrams. Numerous works have suggested a design methodology that enables butterfly-shaped and straight dampers to resist lateral torsional buckling effectively. Likewise, earlier research [20–22] demonstrated that with proper design, the fuses can ensure a correct distribution of shear and flexural stresses throughout the length of the damper. Different studies indicated that the implementation of shear fuses and various dampers could control the structural responses to a significant degree [25–30].

In this paper, the geometric properties are investigated and the possible design ranges are determined to understand the mechanisms of ductile and brittle limit states [13,15,24]. Furthermore, guidelines are established for design strategy suggestions for which the ductile behavior governs the brittle modes for use in various applications. The detailed studies on the limit states and transitions are summarized by the stiffness equation, which can be estimated with the proposed tables. The conventional and new systems are established, and the developed guidelines are then used to compare their behavior.

2. Guideline Design Procedures for Butterfly-Shaped Links

To determine the procedures for redesigning the lateral resisting systems with innovative butterfly-shaped dampers in high-rise buildings, this section is developed according to previous studies on ductile and brittle limit states [12,13,24,25].

2.1. Limit States and Designing Requirements

The mechanical analysis for structural shear links determines suitable energy dissipation for the dampers with the dominating flexural limit state. Desirable performance in dissipation is related to regions of inelasticity; it is best that these regions are located away from areas of discontinuity and geometric variations [26]. The design guidelines are established for the geometric properties that prevent brittle modes and promote ductile behavior.

First, the strategically chosen geometric properties allow for the plastic hinges to be set far from the discontinuous joints, therefore reducing the possibility of the onset of cracks. Next, the energy dissipation studies show sufficient energy dissipation for the dampers with a dominating flexural limit state for a specific amount of steel material compared to the shear dominated dampers [21]. However, for several cases, the space constraints and geometric issues that the flexure-dominated dampers present mean that the shear dominated design could not be used as intended. The flexure and shear capacity are shown in Equations (1) and (2), respectively:

$$P_p^{flexure} = \frac{2n(b-a)at\sigma_y}{L} \tag{1}$$

$$P_p^{shear} = n\frac{\sigma_y at}{\sqrt{3}} \tag{2}$$

Transitional equations to establish the flexural yielding mechanism are shown below. For $a < b/2$:

$$\frac{b-a}{L} < 0.28\ (or\ \alpha > 148^{\circ})\ \text{Flexure dominated} \tag{3}$$

when $a > b/2$ or is slit ($a = b$), the flexure-dominated limit state would initiate for $b/L < 1.15$. Furthermore, the moment and shear capacity limit states for the dampers with $a > b/2$ are assessed with Equations (4) and (5):

$$P_p^{flexure} = \frac{nb^2t}{2L}\sigma_y \tag{4}$$

$$P_p^{shear} = n\frac{\sigma_y bt}{\sqrt{3}} \tag{5}$$

2.2. Brittle and Ductile Mode of Behavior Investigations

The minimum flexure or shear limit states determined with Equation (6) should be less than the buckling limit state determined by Equations (6) and (7) [27].

$$P^p = \min\left\{P_p^{flexure}, P_p^{shear}\right\} \tag{6}$$

$$P_{cr}^{LTB} = \frac{2\mathrm{E}[0.533 + 0.547(a/b) - 0.281(a/b)^2 + 0.096(a/b)^3]bt^3}{L^2\sqrt{1+v}} \tag{7}$$

Studies done by Frazampour and Eatherton show that P^P is the minimum ductile strength and P^{LTB} is the lateral torsional buckling limit state [23]. Hence, the maximum lateral buckling capacity for the brittle mode behavior should have higher values compared to the minimum ductile capacity for each damper. In the event of buckling, it is important to examine the effect of the overstrength factor for the rest of the elements, as shown by Equation (8). Based on the parametric study's post-processing results [24], the overstrength factor could be identified. It is noted for the geometries that b/L is above 0.4; the lower values could be taken into consideration as shown in Table 1, which indicates over-strength factors for various geometric properties of steel dampers.

$$\Omega P^p < P_{cr}{}^{LTB} \tag{8}$$

The average normalized PEEQ values considering the low plastic strain values for hourglass-shaped shear fuses are shown in Table 2. Based on the geometrical investigation, an *a/b* value of 1.0, or 0.1, could lead to high plastic strain concentration values and induce fractures. In general, plastic strain values of 0.33 and 0.75 are suggested for the design and use of various dampers. The high normalized plastic strain values shown in Table 2 indicate less ductility, fracture initiation potential, and resistance issues.

Table 1. The over strength factor for various geometrical properties of steel dampers.

Shear Fuse		b/L	Ω
a/b	0.1	0.1	4.1
		0.2	3.3
		0.3	2.8
		0.4	1.8
	0.33	0.1	2.3
		0.2	1.65
		0.3	1.35
		0.4	1.3
	0.75	0.1	4.13
		0.2	3.18
		0.3	2.51
		0.4	1.95
	1	0.1	4.35
		0.2	3.35
		0.3	2.75
		0.4	2.45

Table 2. The over-strength factors for various geometrical properties of steel dampers.

Normalized PEEQ at 5%	Mid Width (a)/End Width (b)			
b/L	0.1	0.33	0.75	1
0.1	5.82	2.35	2.59	5.18
0.2	8.41	6.18	5.47	15.94
0.3	16.35	1.03	1.00	16.76
0.4	20.71	1.29	1.24	21.76

2.3. Drift Ratio Control Requirement, Based on the Stiffness

Figure 2 depicts the geometric properties of a plate consisting of butterfly-shaped shear links. Taking into account various factors contributing to the total stiffness of a plate with butterfly-shaped links [1,30], the total stiffness of the innovative damper is estimated by Equation (9).

$$K_T = \frac{K_b K_v K_c K_d}{K_b K_v K_c + K_b K_v K_d + K_b K_c K_d + K_v K_c K_d} \tag{9}$$

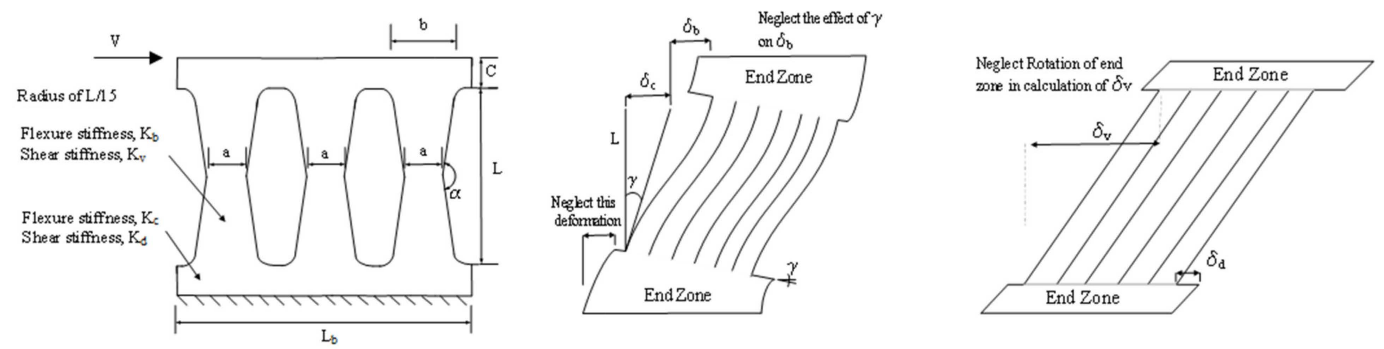
Therefore, the stiffness partial terms of K_b, K_v, K_c, and K_d are derived from Equations (10)–(13):

$$K_b = Eb^3 t \left(\frac{2\left(\frac{a}{b} - 1\right)^3}{3\left[2L_n\left(\frac{a}{b}\right) + \left(\frac{a}{b} - 1\right)\left(\frac{a}{b} - 3\right)\right] L^3} \right) \tag{10}$$

$$K_v = \frac{\frac{5nGt}{6L}(b - a)}{\mathrm{Ln}\left(\frac{b}{a}\right)} \tag{11}$$

$$K_c = n\frac{P}{\delta_c} = \frac{EtL_b{}^3}{6C(L)^2} \tag{12}$$

$$K_d = \frac{5}{6}\frac{GtL_b}{L} \tag{13}$$



(**a**) (**b**) (**c**)

Figure 2. The structural shear damper modes of behavior: (**a**) butterfly-shaped geometry; (**b**) moment deformation; (**c**) shear deformation.

Equations (14) and (15) determine the calculations of the total flexure stiffness ($K_{\text{flexurefactor}}$) and total shear stiffness ($K_{\text{shearfarctor}}$), respectively. These two factors could be interpolated based on data provided in Tables 3 and 4 as well, which summarize the factors for estimating the flexural and shear stiffness values.

$$K_b = \mathrm{n} \times \mathrm{E} \times \mathrm{t} \times \mathrm{K}_{\text{flexurefactor}} \tag{14}$$

$$K_v = \mathrm{n} \times \mathrm{G} \times \mathrm{t} \times \mathrm{K}_{\text{shearfactor}} \tag{15}$$

Table 3. $\mathrm{K}_{\text{shearfactor}}$.

			a/*b*			
			0.1	0.33	0.66	1
K shear factor	*b*/*L*	0.2	0.065	0.100	0.136	0.2
		0.4	0.130	0.201	0.272	0.4
		0.6	0.195	0.302	0.409	0.6
		0.8	0.260	0.402	0.545	0.8

Table 4. $\mathrm{K}_{\text{flexurefactor}}$.

			a/*b*			
			0.1	0.33	0.66	1
K flexure factor	*b*/*L*	0.2	0.2	0.001	0.003	0.005
		0.4	0.4	0.015	0.03	0.047
		0.6	0.6	0.052	0.101	0.159
		0.8	0.8	0.124	0.239	0.378

Slit shear link stiffness could be similarly established by Equation (16):

$$K_T = \frac{K_b K_v K_c K_d}{K_b K_v K_c + K_b K_v K_d + K_b K_c K_d + K_v K_c K_d} \tag{16}$$

Therefore, the stiffness partial terms of K_b, K_v, K_c, and K_d are derived from Equations (17)–(20):

$$K_b = \frac{nEtb^3}{L^3} \tag{17}$$

$$K_v = K_v = \frac{5n}{6}\frac{Gtb}{L} \tag{18}$$

$$K_c = \frac{EtL_b^3}{6cL^2} \tag{19}$$

$$K_d = \frac{GtL_b}{L} \tag{20}$$

The residual displacements are required to be less than the prescribed values for various lateral resisting applications as are determined by Equation (21).

$$\frac{P^{loading}}{K_T} < \delta_{specified} \tag{21}$$

In addition, for applications such as EBF systems, the drift limit state needs to be checked. Considering the AISC 341 drift requirements, the maximum displacement variation is 1 inch for a 12 ft height building. Therefore, Equation (22) could be established.

$$\delta_p = \delta_{up} - \delta_{down} \leq 1 \text{ inch} \tag{22}$$

The rotation angle within the shear links could be calculated and checked according to Equation (23):

$$\gamma_p = \frac{L}{e}\theta_p \text{ where } \theta_p = \frac{\delta_p}{h_{story}} \tag{23}$$

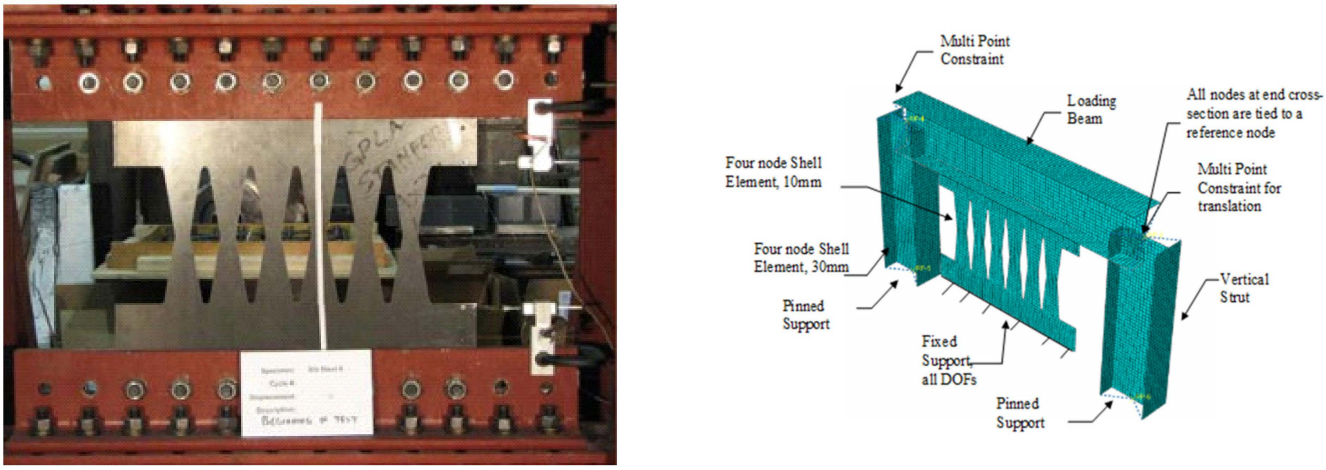
where g_P is the rotation within the link, and θ_p is the story drift angle. The resulting rotation within the link for a 30 ft span is as follows: based on the previous experimental tests, it is shown that the ultimate rotational capacity of butterfly-shaped fuses for various geometries should be less than 0.2 in.

3. Verification of Finite Element Modeling Methodology via Laboratory Testing

To confirm the laboratory testing that observed the yielding and lateral torsional buckling behavior of butterfly-shaped dampers under cyclic loading conditions at different drift intervals, the Finite Element (FE) software utilized. The computational model is built with the same boundary conditions as the laboratory testing, with the edge of the dampers being fully constrained. S4R elements with five integration points are considered to simulate the laboratory test done by Ma et al. [31]. Hour glassing and shear locking events are avoided with the use of hourglass prevention modes and reduced integration points.

To obtain a hysteretic pushover effect, the dynamic explicit solver is utilized to analyze both monotonic and cyclic behavior. The material model used has a yield strength of 273 MPa and an ultimate strength of 380 MPa. The cyclic response is obtained and compared to laboratory test results, as demonstrated in Figure 3.

In order to make appropriate comparisons and obtain similar limit states, a verification study done by Shin et al. [32] is considered. The FE ABAQUS package is implemented to precisely simulate the behavior and capture the post analysis results. A computational model consisting of twenty-point solid elements and reduced integration features is used to prevent the occurrence of shear locking [32]. A bi-linear constitutive material model with a yield strength of 379 MPa, elastic modulus of 200 GPa, and 0.015% hardening is implemented. Based on the testing conditions, the story shear was estimated to be 1.42 times the obtained shear strength, and the drifts are determined based on the chord rotation divided by 1.34. Figure 4 summarizes the shear and drift results under a cyclic loading condition for the hourglass-shaped beam [33]. For the limit state, the buckling occurred at a 2% drift ratio—a value precisely captured from the simulation models. Before and after buckling, the determined strengths from testing were 76 kN and 57.4 kN, while the simulation results determined values of 73.2 kN and 52.7 kN, with less than a 5% margin of difference.



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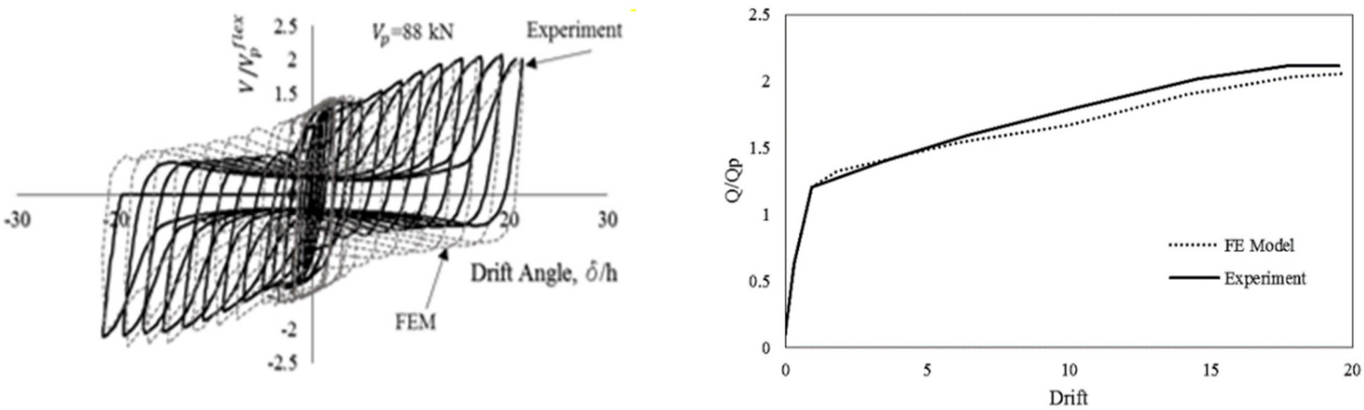
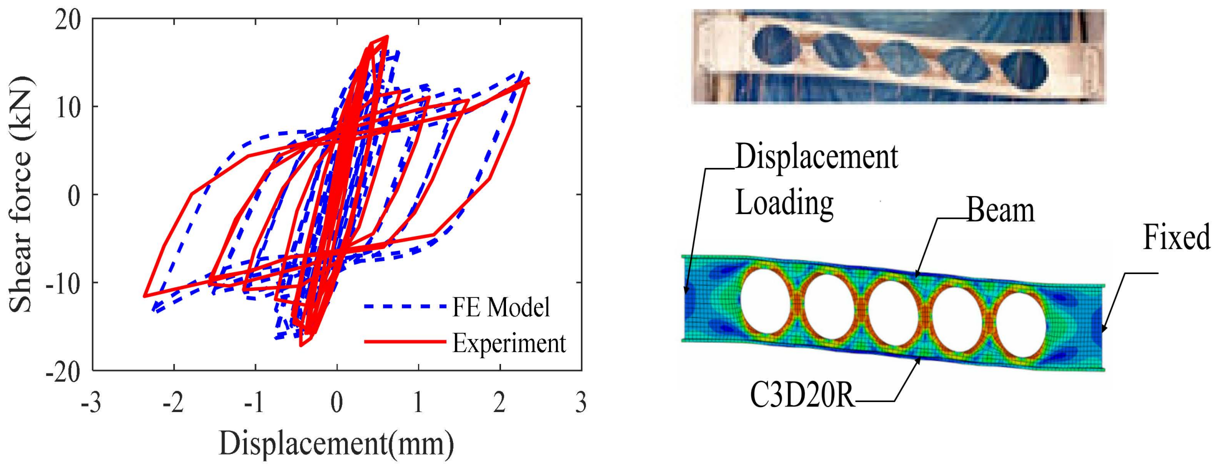


(**b**)

Figure 3. The laboratory test results were confirmed by using a finite element modeling approach: (**a**) FE modeling establishment; (**b**) verification of modeling methodology cyclic and monotonic behaviors.



(**a**) (**b**)

Figure 4. The laboratory test results were confirmed by using a finite element modeling approach: (**a**) verified hysteretic response; (**b**) FE model.

4. Preparation of Reduced-Order Simulation Models for Innovative Lateral Resisting Systems

The reduced-order models used for simulating multi-story buildings based on the primary results of the FE analysis are established by verification of the cyclic behavior. Butterfly-shaped beams are developed accordingly for a conventional EBF prototype system introduced in the IC (2012) code provision for their use in high-rise applications, redesigned with a set of shear fuses and a total beam length of 120 cm.

The details of the computational models are shown in Figure 5; the material model has a yielding stress of 250 MPa, modulus of elasticity of 2×10^5 MPa, and a strain-hardening ratio of 0.0005. To confirm the reduced order model computational system, a cyclic load protocol used in AISC for EBF behavior studies is considered and applied as shown in Figure 5 [33]. The schematic reduced-order model established with Opensees packages is illustrated in Figure 6, where the beams are modeled using elastic elements with negligible effects of the upper and lower steel plates on inelastic behavior.

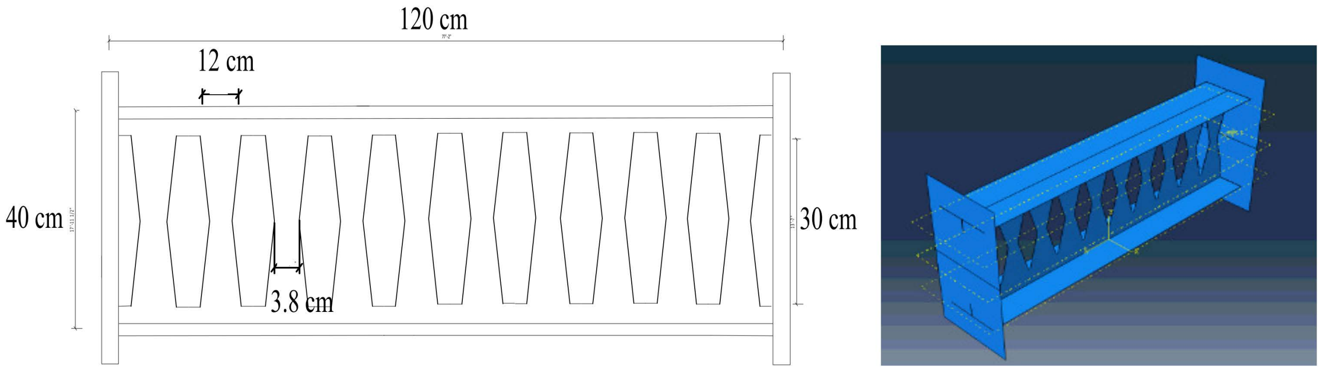
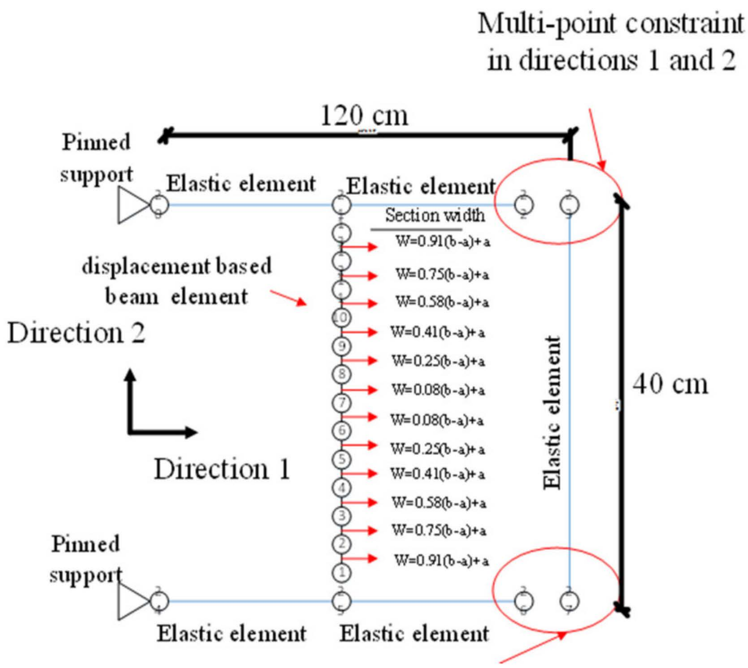


Figure 5. The computational model of the beam.

The displacement-based beam element command (dispBeamColumn) is applied to develop the butterfly-shaped links. The taper-shaped shear links are generated based on the pinned boundary condition, which is schematically shown. The isotropic strain hardening material is developed based on the yielding regime of 248 MPa, a modulus of elasticity of 2×10^5 MPa, and a strain-hardening value of 0.05. From Figure 6, the hysteretic behavior and results of the reduced-order models indicate that the reduced-order model could achieve the hysteretic behavior with 98% accuracy.



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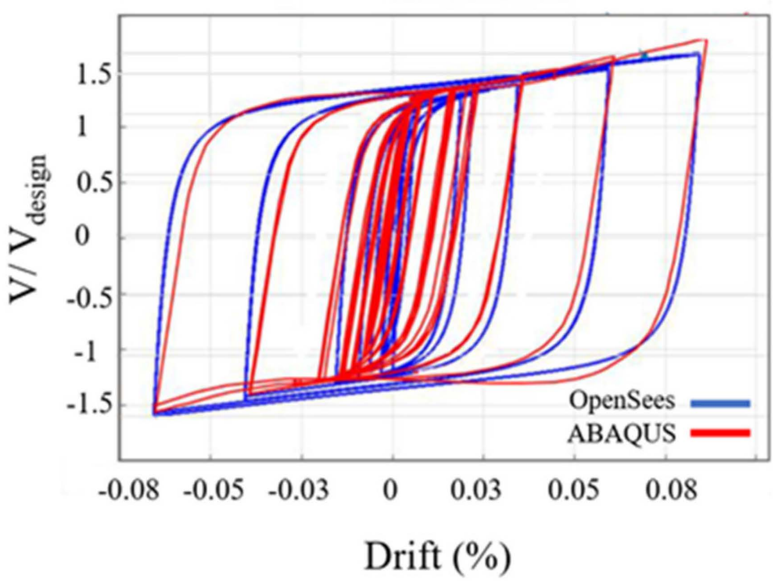

(**b**)

Figure 6. The verified models based on the FE and OpenSees analysis: (**a**) OpenSees reduced-order model schematic illustration; (**b**) reduced-order computational model hysteretic model with a V_{design} of 530 kN.

5. Performance of Multi-Story Prototype Structure Designed Based on the Guidelines

To study the effects of shear link implementations on various multi story buildings, the EBF prototype from SEAOC is selected [34]. By establishing a multistory structure with shear fuses systems, both lateral resisting structures are simulated. To satisfy the equilibrium scenario based on Figure 7, Equations (24) and (25) are applied to determine the forces imposed on the butterfly-shaped damper. Equation (24) is obtained from the general static conditions for the top beam and Equation (25) is developed according to the general equilibrium of the system.

$$V_{BF} = \frac{2M}{H} \;\&\; M = \frac{VL}{2} \quad (24)$$

$$2M - VL + HV_{BF} - V_{BF}H = 0 \quad (25)$$

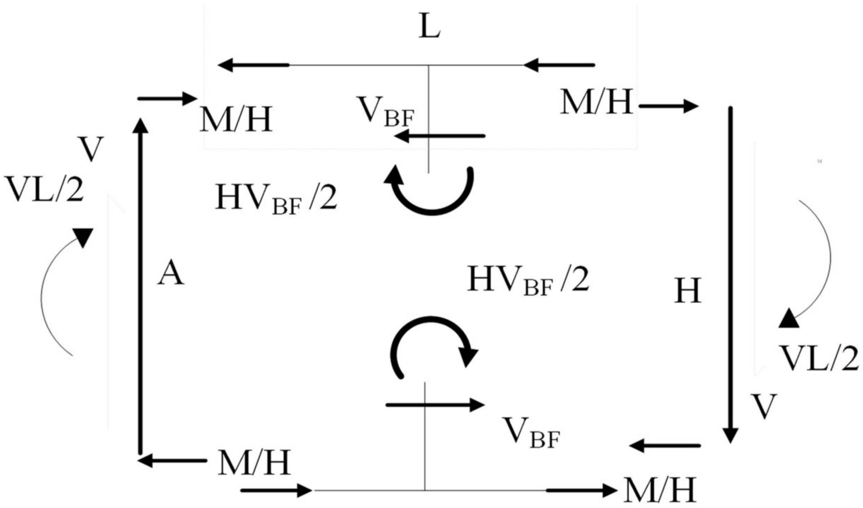

Figure 7. Butterfly shaped damper and the imposed forces on the damper.

By simplifications, Equation (26) could be obtained.

$$V_{BF} = \frac{V \times H}{L} \quad (26)$$

By deriving Equation (26), the design force for the butterfly links could be calculated, as shown in Table 5.

Table 5. The prototype and new lateral resisting systems.

Level	Shear (Kips)	Cumulative Force (kN)	Design Force for the Butterfly Links with Equation (26) (kN)		Design Groups	Specifications
Roof	60	267	1085	1085	III	BU 13 × 53
Six	60	534	1085			BU 13 × 53
Five	88	925	1592	1592	II	BU13 × 53
Four	125	1481	2260	2260	I	W 10 × 68
Third	125	2438	2260			W 10 × 68
Second	125	2593	2260			W 10 × 68

Three design groups are shown in Figure 8. According to guidelines, the fuses are established to accommodate and meet the same demand forces reported by SEAOC that are used for establishing the typical conventional EBF systems [34].

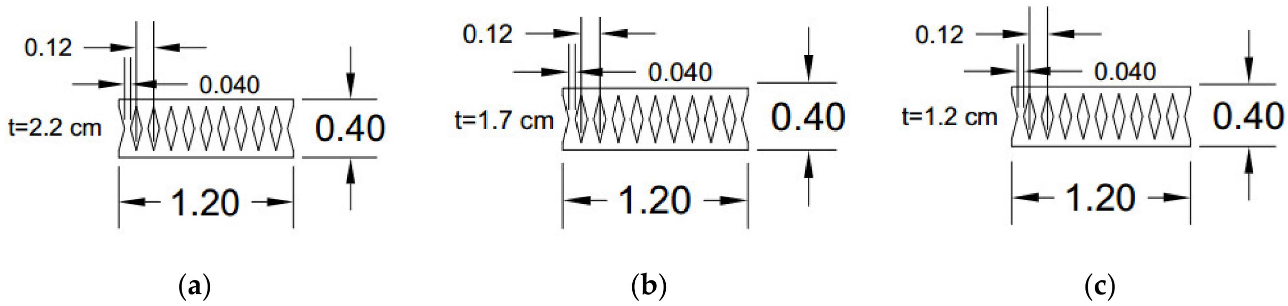

Figure 8. Three design groups used, developed according to guidelines (thickness in centimeter): (**a**) Group I; (**b**) Group II; (**c**) Group III.

If ductile mechanisms are intentionally redirected from tension-induced buckling, the energy dissipation can be improved. The study investigates the impact of this innovative damper on various prototype buildings and compares it to traditional systems. By using finite element models to study different prototypes, the mechanics, equations, and design tools are developed to allow for the tuning of lateral resistance behavioral features, such as stiffness and strength.

The fuse requirements are in accordance with the provided guidelines in previous sections. Figure 8 shows the six-story simulations with the links. The schematic conventional and butterfly models are shown in Figure 8a. The details of the computational models considering the leaning column effects to simulate P-D effects are shown in Figure 8b,c. The gravity loads are estimated according to the total weight of the structure related to the seismic performance and are imposed at each story, which is subsequently divided by two to account for each column in the lateral resisting EBF system [33].

The yielding mechanisms in discrete steel segments can be controlled by setting a limit on the buckling state, leading to flexural and shear stresses. By designing new concepts, energy dissipation capability can be improved and demands on structural boundary elements can be reduced in steel structural applications. The damping ratio is determined using Raleigh ratio recommendations for the first couple of natural modes, and is found to be 0.02. In conventional structures, the linking beam is designed to have a shear yielding mechanism. To achieve this, a zero-length element is used in the Opensees software package to establish the shear stiffness of the system in the vertical direction, as illustrated in Figure 8. The models considered for NRHA studies in Figure 9.

The allowable drift ratio is determined for each story in the two models, according to the design forces. All the drift ratios are shown to be less than 0.02 off the drift ratio code requirement limit. Based on further investigation between the conventional and butterfly equipped system, it is found that both systems initiate yielding within a similar strength capacity range, while the conventional systems could not achieve the larger drift ratios compared to the butterfly fuse equipped systems shown in Figure 10. This study summarizes the step-by-step procedure to establish a new generation of lateral resisting systems utilizing various dampers. Future studies will be allocated to the investigation of the performance of the conventional systems and the systems with the new generation of dampers. The analysis suggests that, in numerous seismic shear damper systems, the yielding starts within the damper, and the plastic strains accumulate in the damper rather than the plates.

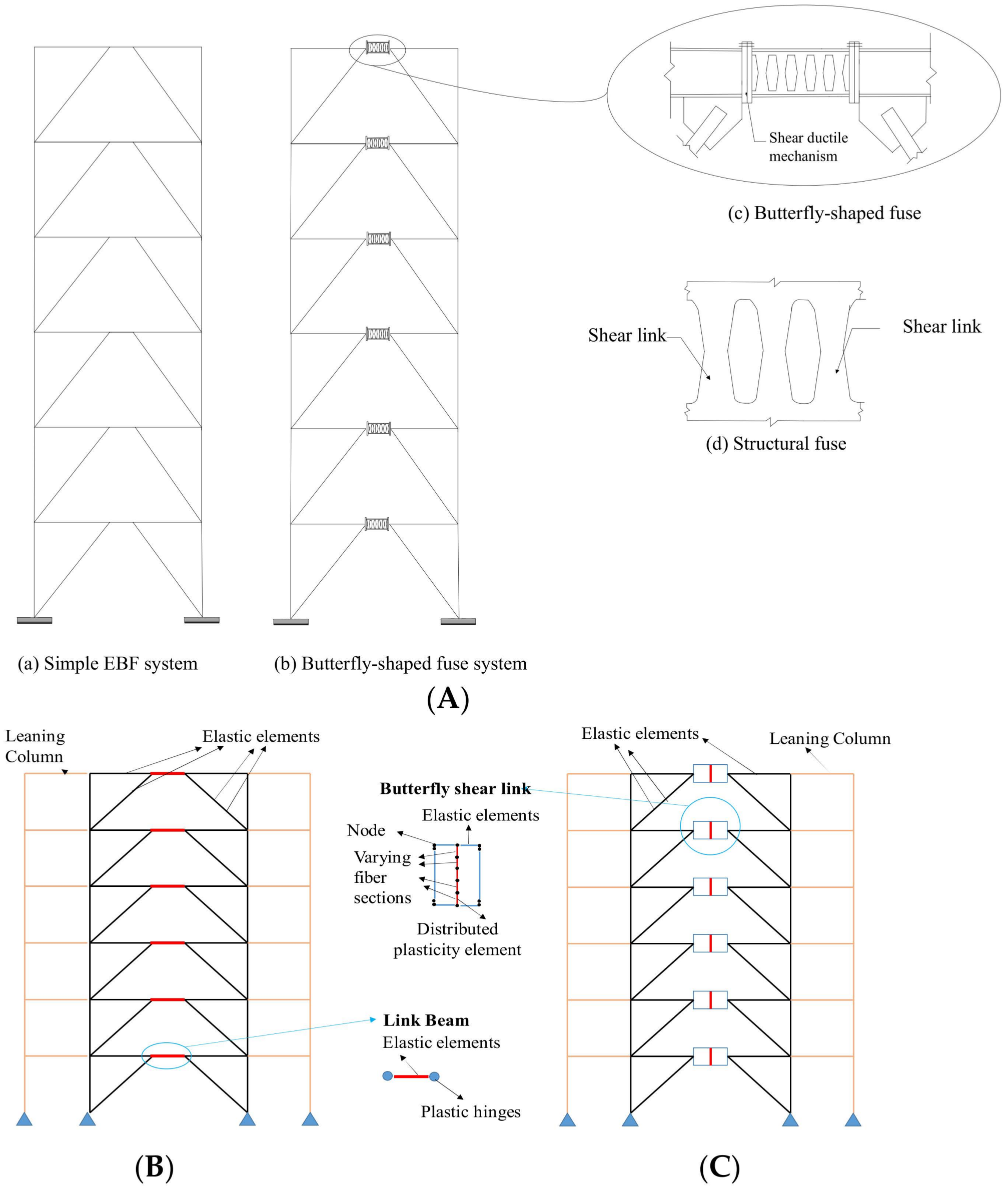

Figure 9. The models considered for NRHA studies: (**A**) the schematic representation of new generation of dampers used for EBF structural systems; (**B**) the model with the damper; (**C**) the conventional system.

This study examines and compares the implementation of a new lateral resisting system, which includes butterfly-shaped dampers, with the EBFs in multi-story prototype buildings. The new design converts global shear deformations into local flexural yielding mechanisms, which can reduce demands on other structural boundary elements and improve seismic performance. The new system is shown to have similar stiffness and strength as the conventional system, but with over 30% lower demands on boundary elements, leading to more efficient and economical design procedures for seismic intensive areas.

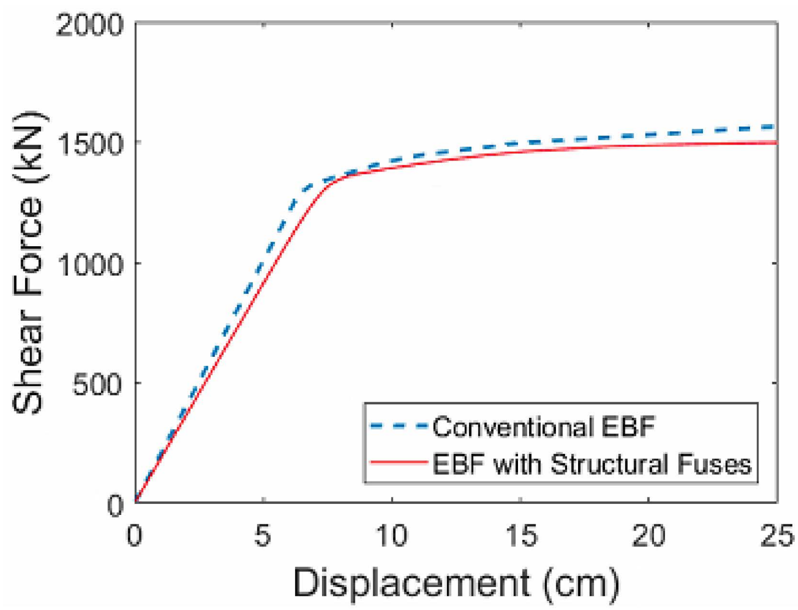

Figure 10. A comparison of the pushover curves is made between the conventional EBF system and the new lateral resisting system that includes structural fuses.

6. Conclusions

Shifting towards the ductile modes of behavior, where buckling and early tension yielding and local flexure or shear yielding occur, could improve seismic resistance capability and energy dissipation. Structural fuses are utilized to focus damages in specific areas of buildings, which has multiple benefits in reducing their vulnerability to seismic activity. These systems are effective at preventing stiffness and strength degradation, resulting in more stable hysteretic responses. Additionally, the use of structural fuses can protect nearby elements from excessive force and inelasticity, providing further safeguards against seismic damage.

The commonly used conventional EBFs are investigated and redesigned with the system of innovative dampers. Initially, seismic dampers used in building applications Were studied based on prototype buildings by implementing the verified simulations. Next, seismic protective fuse systems were introduced, and design requirements were examined. Guideline design procedures have been established and the EBF systems have been redesigned.

The effects of the shear links on multi-story prototype buildings and conventional systems have been displayed. It is concluded that the innovative and strategically designed configurations that convert the global shear deformations into ductile modes of behavior could have practical advantages for steel systems as they improve various structural capabilities and demand less boundary elements. The prototype building application can withstand early brittle limit states and dissipate energy more efficiently by regulating the occurrence of flexural and shear yielding mechanisms throughout the steel segments.

Author Contributions: Conceptualization, A.F., I.M., J.-W.H. and S.J.M.; methodology, A.F.; Investigation, A.F.; Data curation, M.R.K.; writing—original draft, A.F. and S.J.M.; writing—review and editing, A.F., I.M., J.-W.H., S.J.M. and E.R.; funding acquisition, A.F.; resources, I.M. and J.-W.H.; supervision, J.-W.H. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by Post-Doctoral Research Program for Excellence Institute (2023) in the Incheon National University.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Acknowledgments: This work was supported by Post-Doctoral Research Program for Excellence Institute (2023) in the Incheon National University.

Conflicts of Interest: The authors declare no conflict of interest.

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