Research Article

Seismic Behavior of Box Steel Bridge Pier with Built-In Energy Dissipation Steel Plates

Xue Han,1 Anlong Jiang,2 Jinhuo Zheng,3 Xiangui Zhou,4 Yewei Chen,5 Sibao Fang,6 Xin Li,3 Jianhua He,7 and Haifeng Li 8

1Xiamen Institute of Technology, Xiamen, China
2Xiamen Road and Bridge Engineering Design Institute Co., Ltd, Xiamen, China
3Fujian Provincial Institute of Architectural Design and Research Co., Ltd., Fuzhou, China
4Hangzhou Municipal Construction Group Co., Ltd., Hangzhou, China
5Construction & Development Co., Ltd., China Construction Fourth Bureau, Xiamen, China
6Shanghai Baoye Group Co., Ltd., Shanghai, China
7Fujian First Highway Engineering Group Co., Ltd., Quanzhou, China
8College of Civil Engineering, Huaqiao University, Xiamen, China

Correspondence should be addressed to Haifeng Li; lihai_feng@126.com

Received 6 July 2022; Revised 27 September 2022; Accepted 8 October 2022; Published 21 October 2022

Academic Editor: Daniela Pilone

Copyright © 2022 Xue Han et al. This is an open access article distributed under the Creative Commons Attribution License, which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

Based on the design concept of earthquake-resilient structure, a new-type of box-shaped steel piers with embedded energy-dissipating steel plates was proposed. Quasi-static tests of 6 box-shaped steel pier specimens under variable axial pressure and cyclic horizontal loading were carried out. By analyzing the failure mode, load-displacement hysteretic curve, skeleton curve, displacement ductility coefficient, stiffness degradation characteristics, strength degradation coefficient, and cumulative hysteretic energy, the effects of setting energy-dissipating steel plate, axial compression ratio, and thickness of energy dissipation steel plates on the seismic performance of new-type steel piers were discussed. Finite element models of steel bridge piers were established and compared with the test results. The analysis results using FEM agree well with the test results. Results show that the setting of energy-dissipating steel plates can effectively improve the ductility, deformation capacity, and energy-dissipating capacity of box-shaped steel piers, and effectively delay buckling deformation and cracking of wall plates. The steel plate near the bolt hole of the wall plate at the root of the new-type of box-shaped steel piers is easy to crack due to stress concentration, resulting in a rapid reduction of the maximum bearing capacity of the specimens. With the increase of axial compression ratio, the bearing capacity, energy-dissipating capacity, and earthquake-resilient capacity of the specimens increase. The smaller the thickness of the replaceable energy-dissipating steel plates, the smaller the bearing capacity and faster the stiffness degradation of the specimens become, while the ductility and energy-dissipating capacity of the specimens are improved. The axial compression ratio and the thickness of the energy-dissipating steel plate have relatively little effect on the strength degradation of the specimens. In order to facilitate the popularization and application of the new-type steel piers, formulas were also established to calculate the bearing capacity and displacement ductility factor of the new-type of box-shaped steel piers.

1. Introduction

Reinforced concrete piers have been widely accepted as robust bridge supports characterized by their notable size and self-weight, limited ductile behavior resulting in undesired seismic responses, serious consequences, and heavy rehabilitation. Comparably, steel piers have the merits of lighter weight, easier construction, and maintenance, suggesting greater potentials for general urban bridges in China. Moreover, steel piers tend to deliver desired ductile behavior in terms of well dissipating seismic energy, preventing brittle failure, and producing a resilient system to protect personal safety. However, a ductile behavior generally suggests inevitable capacity loss which requires a quick rehabilitation to keep the bridge safe [1–3].
Many experimental and numerical studies have been conducted to investigate the yielding steel introduced energy-dissipating behavior of steel piers. Then, rehabilitations can be effectively applied for damaged elements to deliver strengthened piers with desired capacity and failure modes. Ismail et al. reveal the impacts of slenderness on enhancing the ductile behavior of box-shaped piers under seismic load, which may result in a completed capacity loss in extreme scenarios [4]. A greater ductile factor is therefore suggested for more slender reinforced ribs. The study of nonlinear behavior of steel piers under bidirectional ground movement demonstrates the impact of radial and circumferential components on mechanical energy [5]. Other factors that affect the ductile behavior of steel piers include material and geometrical properties [6–10] and axial force fluctuations [11–13]. On the other hand, efforts have been also made to improve the ductile behavior. The application of built-in cruciform plates is able to deliver better performance in terms of ductility and energy absorption than ordinary piers [14]. Internal strengthening devices have been successfully applied to improve energy dissipation capacities resulting in a slower deterioration [15]. Seismic dampers and shock transmissions are made to effectively isolate seismic load [16], which is found successfully to mitigate residual deformation. Energy can also be well dissipated by yielding the core steel of braces through restraint of compression buckling [17]. Various other bracing systems have also been made to successfully dissipate seismic energy via friction devices and/or a self-centering system forcing structures back to the original position after earthquakes [18, 19].

Meanwhile, more studies have been conducted upon the structural fuse concept in which easily replaceable elements are applied to well dissipate seismic energy, keeping the structure elastic [20]. Replaceable links with section, connection, and welding details are proposed to develop stable and repeatable yielding elements for shear reinforcements [21, 22]. For example, replaceable dissipaters are developed to deliver stable energy dissipation and re-centering properties for unbounded post-tensioned bridge piers [23]. In addition, shape memory alloy (SMA) washers are used at the footings of RC piers to improve the self-centering capability and energy dissipation of bridges [24]. Other replaceable elements including steel coupling beams with specific connections [25] and low-yield-point connections of steel frames [26], are able to cost-effectively enhance the post-damage capacity.

Upon the existing outcomes, this study proposes a novel built-in box-shaped steel pier having specific replaceable elements to deal with various damages, which suggests a quick, cost-effective, and comprehensive methodology for seismic rehabilitation. Experimental studies in the usage of six box-shaped specimens have been tested to investigate the impacts of axial compression ratio, plate thickness, configuration of dissipating plates on the quasi-static behavior, failure modes, ductility, stiffness and strength degradation, and energy-dissipating capacity. Then, design recommendations are proposed to guide the usage of the brand-new piers delivering desired seismic behavior and rehabilitation solutions.

2. Experimental Program

The proposed pier consisted of a loading component, an energy-dissipating zone, and a rigid support as shown in Figure 1. More configuration details can be found in Figure 2. The pier was made to develop a plastic hinge from the critical moment and shear at the end. Additional reinforcements in terms of transverse diaphragms and longitudinal ribs were applied to guide the major deformation occurred in the energy-dissipating zone. Then, replaceable components consisting of one Q355 steel plate, two Q355 steel chams, and high-strength bolted connections were applied to dissipate the seismic energy in the specific zone. The bolted connections not only offered a quick replacing solution but also were capable of well-fastening steel plates to resist the notably seismic load. Plastic deformations were therefore expected to first occur in those plates which well dissipated seismic energy to protect the main structure of the pier. Steel clamps were applied to restrain lateral buckling for making fuller usage of energy-dissipating plates. The cores of energy-dissipating plates were made of lower-strength steel to effectively attract major deformation. Both the plates and the entire energy-dissipating zone were replaceable to better meet various rehabilitation needs after earthquakes.

2.1. Specimens. A scale of 1 : 4 was applied for six box-shaped piers which had various thicknesses of energy-dissipating plates, and axial compression ratios as listed in the I, II, and III groups of Table 1. High-strength bolts were applied to connect three parts, i.e., the loading component, the energy-dissipating zone, and the rigid support. The cross section of all specimens was 370 × 350 mm in which all side plates were made of Q355 steel. The energy-dissipating zone consisted of two Q355 clamp plates and one Q235 core which were connected to the side plates of box-shaped piers by M12 bolts. Group I consists of a control specimen I-0 in the usage of ribs to strengthen the section corner. In groups II and III, high-strength M12 bolts were used to connect energy-dissipating plates and clamps to the specific zone, and ribs were applied for the corners as well. All parameters except the plate thickness are kept the same for specimen II-1 and II-2 in Table 1. Similarly, specimens III-1, III-2, and III-3 were directly comparable tests with various axial compression ratios. More details can be found in Table 1.

2.2. Test Setup. In the test setup, a 100t hydraulic ram was used to produce vertical loads and a 100t MTS testing machine was used to control lateral displacement, as shown in Figure 3. The literature [27] deeply analyzed the loading protocols of traditional and novel seismic-resilient steel frame members, revealing the variations in critical seismic demands imposed on different systems under different types of earthquakes. The research results enrich the existing loading protocols of quasi-static tests. However, the test loading protocol in this paper mainly refers to the ‘Code for seismic design of buildings: GB 50011–2010 [28] and Specification for seismic test of buildings: JGJ/T 101–2015.
Lateral displacements were $\pm 0.5\delta_y$ and $\pm 0.75\delta_y$ for the preloading process, in which $\delta_y$ represented the yield displacement according to numerical calculations. When the specimens were loaded with an increment of one $\pm \delta_y$ after 3 cycles till the ultimate failure. More loading details can be found in Figure 4 and Table 2.

![Figure 1: New type of box-shaped steel pier. (a) Three-dimensional diagram. (b) Energy-dissipating zone.](image1)

![Figure 2: Details of the new box-shaped steel pier/mm. (a) Elevation. (b) Cross section of group I (c) Cross section of group II, III.](image2)

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimen heights $H$ (mm)</th>
<th>Heights of energy-dissipating zone $L$ (mm)</th>
<th>Thicknesses of side plates $t$ (mm)</th>
<th>Thicknesses of energy-dissipating plates $t_1$ (mm)</th>
<th>Thicknesses of clamps $t_2$ (mm)</th>
<th>Axial compression ratios $n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-0</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>—</td>
<td>—</td>
<td>0.20</td>
</tr>
<tr>
<td>II-1</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>4</td>
<td>6</td>
<td>0.20</td>
</tr>
<tr>
<td>II-2</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>0.20</td>
</tr>
<tr>
<td>III-1</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>8</td>
<td>6</td>
<td>0.20</td>
</tr>
<tr>
<td>III-2</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>8</td>
<td>6</td>
<td>0.20</td>
</tr>
<tr>
<td>III-3</td>
<td>2635</td>
<td>1000</td>
<td>6</td>
<td>8</td>
<td>6</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Note. $n = N/f_yA$; $N$ is axial compression load; $f_y$ is yield strength of steel material; $A$ is the cross-sectional area of box-shaped pier.

[29] Lateral displacements were $\pm 0.5\delta_y$ and $\pm 0.75\delta_y$ for the preloading process, in which $\delta_y$ represented the yield displacement according to numerical calculations. Then, the specimens were loaded with an increment of one $\pm \delta_y$ after 3 cycles till the ultimate failure. More loading details can be found in Figure 4 and Table 2.
2.3. Measurements. Lateral loads were transversely applied, i.e., in the north-south direction, as shown in Figure 5, to produce positive displacements while the loading part moved towards the south. Negative displacements were therefore defined as the load moved towards the north. Cells were applied to record both vertical and lateral loads. A laser measurement SJ1 was applied to record the lateral displacement at the tops. Lateral measurements were used to record the side displacement in the north and west directions, respectively. The displacements of energy-dissipating plates were also measured. Measurement SJ7 and SJ8 were applied for recording the lateral displacement of supports. Four more measurements SJ9, SJ10, SJ11, and SJ12 were used to identify any vertical displacement for the rigid support.

![Figure 3: Test setup. (a) Three-dimensional diagram. (b) Elevation diagram. (c) Photograph of test.](image)

![Figure 4: Loading procedure.](image)

<table>
<thead>
<tr>
<th>No.</th>
<th>Loading heights $L_1$ (mm)</th>
<th>Yield displacements $\delta_y$ (mm)</th>
<th>Axial loads $N$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-0</td>
<td>1880</td>
<td>10.50</td>
<td>586</td>
</tr>
<tr>
<td>II-1</td>
<td>1880</td>
<td>10.50</td>
<td>586</td>
</tr>
<tr>
<td>II-2</td>
<td>1880</td>
<td>10.50</td>
<td>586</td>
</tr>
<tr>
<td>III-1</td>
<td>1880</td>
<td>10.50</td>
<td>586</td>
</tr>
<tr>
<td>III-2</td>
<td>1880</td>
<td>11.82</td>
<td>293</td>
</tr>
<tr>
<td>III-3</td>
<td>1880</td>
<td>9.85</td>
<td>732</td>
</tr>
</tbody>
</table>
Figure 5(c) demonstrates the arrangements of strain gauges. South and east sides had a total of 16 strain gauges. Another six gauges and ten sets of rosette strain gauges were applied to measure the deformation in the north and west sides. All details can be found in Figure 5 in which \( P \) represents strain gauges, and \( H \) is used for rosette strain gauges.

### 2.4. Material Properties

According to the standard of GB/T 228.1–2010 [31], direct tensile tests were conducted for Q235 and Q355 steel coupons. The coupon details are shown in Figure 6. Three nominal thicknesses (i.e., 4 mm, 6 mm, and 8 mm) were prepared for Q235 coupon tests, in which all parameters except the thickness were kept the same. Another three nominally identical coupons were applied to identify the tensile strength of energy-dissipating parts in the usage of Q355 steel. Experimental results are listed in Table 3.

### 3. Test Result

#### 3.1. Failure Modes

As shown in Figures 7–12, three major failures have been observed from in test results. In the category A, severe buckling side plates at the end of the energy-dissipating zone resulted in the ultimate failure. Minor cracks were also observed at the side plates, suggesting a notable ductile failure, as shown in Figure 7 (specimen I-0). The category B featured a crack extending from bolt holes to splitting off the side plates at the end of the energy-dissipating zone. No notable buckling deformation was observed, suggesting shear failure dominating specimens II-1 and II-2, as shown in Figures 8 and 9. More comprehensive failure has been found from category C in which minor buckling deformation and severe crack leading to splitting off the side plates were observed at the end of the energy-dissipating zone. All tests in the group III eventually failed in shear, as shown in Figures 10–12.

Without energy-dissipating plates, specimen I-0 developed considerable deformation under the combination of both vertical and lateral loads. On the other hand, the application of energy-dissipating plates mitigated and delayed the buckling deformation of the specimens in groups II and III. This clearly demonstrates the great performance of the energy-dissipating system, which improved the deformation capacity by mitigating the buckling failure at the end. The direct comparisons within group II suggest that increasing plate thickness has no significant impact on the displacements at the buckling, first crack, and ultimate failure. Instead, notably fewer splitting-off cracks were found from 8 mm-thickness plates than that of 4 mm- and 6 mm-thickness counterparts, suggesting the merits of a thicker energy-dissipating plate in delaying crack extending and therefore dissipating seismic energy more effectively. The
A direct comparison within group III indicates that increasing the axial compression ratio tended to reduce the corresponding displacements at the buckling, the cracking, and the ultimate stage. A greater ratio of axial compression also performed well to reduce the ductile deformation of energy-dissipating plates, shifting the mode into shear failure.

3.2. Load-Displacement Responses. Figure 13 demonstrates the load-displacement hysteretic responses, in which $\delta$ is the resulted displacement from the lateral load $V$ applied on the top. The wide and full shape of load-displacement curves suggests that all specimens performed well to dissipate dynamic energy.

With energy-dissipating plates, greater mean displacements at the yielding, maximum, cracking, and ultimate load were observed in group II and III than that in group I. More notably lateral displacements were also observed in the hardening stage. Those observations clearly demonstrate the capacity of energy-dissipating plates on delaying cracking and enhancing deformation capacity. It should be noted that Figures 13(a)–13(b) show a slightly compromised reverse...
capacity of 6.7% compared with that of the control test. This might come from the predrilled hole-induced stress concentration which eventually tore apart the plates before buckling. The performance of predrilled holes is therefore able to determine the capacity of side plates, suggesting their impacts on the ultimate capacity of box-shaped piers. Increasing the plate thickness produced more comprehensive failure modes from wider and fuller energy-dissipating...
Figure 10: Failure mode of specimen III-1.

Figure 11: Failure mode of specimen III-2.

Figure 12: Failure mode of specimen III-3.
Figure 13: Load-displacement curves. (a) Specimen I-0 \((n = 0.2)\). (b) Specimen II-1 \((n = 0.2, t_1 = 4 \text{ mm})\). (c) Specimen II-2 \((n = 0.20, t_1 = 6 \text{ mm})\). (d) Specimen III-1 \((n = 0.20, t_1 = 8 \text{ mm})\). (e) Specimen III-2 \((n = 0.1, t_1 = 8 \text{ mm})\). (f) Specimen III-3 \((n = 0.25, t_1 = 8 \text{ mm})\).

Figure 14: Continued.
shapes, as shown in Figures 13(b)–13(d). Specimen III cracked at the third cycle of 4\(\delta_y\) while the directly comparable specimens II-1 and II-2 using thinner plates developed cracks at the first cycle of 4\(\delta_y\). This suggests the merits of a thicker plate in mitigating cracking.

3.3. Skeleton Curve. Figure 14(a) shows the direct comparison between specimen III-1 and I-0, i.e., specimens with/without energy-dissipating plates. Few differences are observed at the elastic stage, suggesting the limited impact of energy-dissipating plates on the elastic behavior. The application of energy-dissipating plates even reduced the maximum loads by 3.9% and 5.5% in the positive and negative directions, respectively, suggesting a capacity loss. Instead, greater deformation capacities were observed from the direct comparison of specimens I-0 and III-1. The control test reached the ultimate at 4\(\delta_y\), while a much greater displacement of 5\(\delta_y\) was required to fail the counterpart specimen III-1.

Figure 14(b) shows the direct comparison of specimens having various plate thicknesses, i.e., specimen II-1 with a thickness of 4\(\text{mm}\), specimen II-2 having a thickness of 6\(\text{mm}\), and specimen III-1 using a thickness of 8\(\text{mm}\). With a thicker plate, the slightly stiffer specimen III-1 produced a notably greater slope than that of specimens II-1 and II-2. A notable drop was observed as cumulative damages and greater cracking produced the ultimate failure. Thicknesses of 4\(\text{mm}\) and 8\(\text{mm}\), i.e., specimens II-1 and III-1, resulted in a sudden drop compared with that of specimen II-2 having a thickness of 6\(\text{mm}\), suggesting a limited impact of plate thickness on strength degradation. A comparable thickness of the energy-dissipating plate to the side steel tended to

<table>
<thead>
<tr>
<th>No.</th>
<th>Specimens</th>
<th>Loading direction</th>
<th>Yielding points</th>
<th>Maximum points</th>
<th>Ultimate points</th>
<th>Ratio of ultimate to yielding (\mu)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-0</td>
<td>Positive</td>
<td>10.50 1/190</td>
<td>41.90 1/48</td>
<td>63.32 1/32</td>
<td>6.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>–10.50 1/190</td>
<td>–53.53 1/37</td>
<td>–63.40 1/32</td>
<td>6.04</td>
<td></td>
</tr>
<tr>
<td>II-1</td>
<td>Positive</td>
<td>10.50 1/190</td>
<td>52.52 1/38</td>
<td>52.52 1/36</td>
<td>5.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>–10.50 1/190</td>
<td>–52.63 1/38</td>
<td>–52.56 1/38</td>
<td>5.01</td>
<td></td>
</tr>
<tr>
<td>II-2</td>
<td>Positive</td>
<td>10.50 1/190</td>
<td>52.20 1/38</td>
<td>52.78 1/38</td>
<td>5.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>–10.50 1/190</td>
<td>–51.71 1/39</td>
<td>–52.62 1/38</td>
<td>5.01</td>
<td></td>
</tr>
<tr>
<td>III-1</td>
<td>Positive</td>
<td>10.50 1/190</td>
<td>52.34 1/38</td>
<td>62.99 1/32</td>
<td>6.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>–10.50 1/190</td>
<td>–53.12 1/38</td>
<td>–63.03 1/32</td>
<td>6.00</td>
<td></td>
</tr>
<tr>
<td>III-2</td>
<td>Positive</td>
<td>11.82 1/169</td>
<td>47.20 1/42</td>
<td>59.18 1/34</td>
<td>5.01</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>–11.82 1/169</td>
<td>–47.43 1/42</td>
<td>–59.12 1/34</td>
<td>5.00</td>
<td></td>
</tr>
<tr>
<td>III-3</td>
<td>Positive</td>
<td>9.85 1/203</td>
<td>37.57 1/53</td>
<td>49.66 1/40</td>
<td>5.04</td>
<td></td>
</tr>
</tbody>
</table>

Note. \(\theta_y = \delta_y/L_1\); \(\theta_u = \delta_u/L_1\); \(L_1\) is shown in Figure 2(a).
have more stable behavior under cycling loads. Although specimen III-1 in the usage of an 8-mm-thickness plate compromised around 5% maximum load in the positive direction compared with specimen II-1 having a 4-mm-thickness plate, a greater ultimate displacement of $6\delta_y$ was reached. The comparable ultimate displacement of specimens II-1 and II-2 was $5\delta_y$. This observation indicates that a thicker plate helps to develop greater deformation capacity. The impacts of axial compression ratio are shown in Figure 14(c), in which three representative specimen III 1–3 in the usage of ratio 0.2, 0.1, and 0.25, respectively. With a greater ratio, specimens III-1 and III-3 developed a stiffer slope than specimen III-2. The greatest ratio of 0.25 even produced an 11.2% larger load in the positive direction than that of specimen III-2 having the smallest ratio of 0.1. A larger ratio tended to result in a greater ultimate displacement. Increasing the ratio from 0.1 to 0.2 produced a notably greater displacement $6\delta_y = 62.99$ mm than that of the counterpart $5\delta_y = 49.24$ mm at the ultimate. However, a further increment of the ratio from 0.2 to 0.25 resulted in less notable improvement in terms of displacement $5\delta_y = 59.08$ mm.

3.4. Displacement Ductility Coefficient. A diagram method was applied to determine the nominally yielding point for the skeleton curve. The corresponding displacement to the
point was therefore defined as nominally yielding displacement $\delta_y'$. The ultimate displacement $\delta_u$ occurred while the applied load dropped to 85% of the maximum load. The displacement ductility coefficient $\mu$ [32] was applied to evaluate the deformation capacity:

$$\mu = \frac{\delta_u}{\delta_y'}.$$  \hspace{1cm} (1)

Table 4 lists the test results in terms of $\delta_y'\delta_u\delta_{\text{max}}$ and $\mu$. Without energy-dissipating plates, specimen I-0 developed a greater value of $\delta_u$ and $\delta_{\text{max}}$ which resulted from a greater buckling deformation occurred at the end. Comparably, stress concentrated around the hole area and a premature crack developing along the welding connection compromised both the load and deformation capacity of specimen II-1. Increasing the plate thickness from 4 mm to 6 mm tended to produce a greater value of $\delta_u$ and $\mu$. This indicates the impacts of plate thickness on the ductility and deformation capacity. The direct comparison between specimens III-1 and III-2 might suggest that the value of $\mu$ could be reduced by an increased ratio of axial compression. However, specimen III-3 having the smallest ratio produced the least value of $\mu$, which might be a direct result from the reduced scale aggravating the stress concentrated around holes and the pre-existing deficiency because of welding. Specimen III-3 was therefore considered failing in shear and developing the least ductility.

In addition, it can be seen from Table 4 that when the thickness of the energy dissipation plate increases from 0 mm to 6 mm, the ultimate drift ratio of the specimen
Figure 17: Cumulative hysteretic energy curves. (a) Impacts of energy-dissipating plates. (b) Impacts of the thickness of energy-dissipating plates. (c) Impacts of axial compression ratio.

<table>
<thead>
<tr>
<th>No.</th>
<th>n</th>
<th>$t_1$ (mm)</th>
<th>$\mu$</th>
<th>$\mu_1$</th>
<th>Errors</th>
</tr>
</thead>
<tbody>
<tr>
<td>II-1</td>
<td>0.2</td>
<td>4</td>
<td>5.01</td>
<td>5.00</td>
<td>0.00%</td>
</tr>
<tr>
<td>II-2</td>
<td>0.2</td>
<td>6</td>
<td>5.02</td>
<td>5.02</td>
<td>0.00%</td>
</tr>
<tr>
<td>III-1</td>
<td>0.2</td>
<td>8</td>
<td>6.00</td>
<td>5.55</td>
<td>-7.48%</td>
</tr>
<tr>
<td>III-2</td>
<td>0.1</td>
<td>8</td>
<td>5.01</td>
<td>5.01</td>
<td>0.00%</td>
</tr>
<tr>
<td>III-3</td>
<td>0.25</td>
<td>8</td>
<td>5.02</td>
<td>5.41</td>
<td>7.77%</td>
</tr>
</tbody>
</table>

Note. $\mu$, $\mu_1$ represent measured and calculated displacement ductility factors, respectively.
gradually decreases, indicating that the reasonable thickness of the energy dissipation shell plate can effectively reduce the residual deformation of the pier. When the axial compression ratio of the specimen increases from 0.1 to 0.25, the ultimate drift ratio of the specimen gradually decreases, indicating that increasing the axial compression ratio can further reduce the residual deformation of the pier.

3.5. Stiffness Degradation. Figure 15 shows the stiffness degradation of all specimens under cycling loads. The degradation was represented by the secant-stiffness ratio of the \( n \)th cycle \( K_n \) to the first cycle \( K_1 \). The corresponding value of \( \delta/\delta_n \) was the displacement ratio of the given cycle \( \delta \) displacement to the first cycle \( \delta_1 \) displacement. The value of \( K_n \) can be obtained by using the following equation.

\[
K_n = \frac{|V_n| + |V_1|}{|\delta_n| + |\delta_1|},
\]

\( V_n \) is the lateral load at the \( n \)th cycle, \( \delta_n \) is the resulted displacement from the maximum lateral load at the \( n \)th cycle.

As shown in Figure 15(a), the control specimen I-0 developed a greater initial stiffness than that of specimen II-1, suggesting the negative impacts of energy-dissipating plates on compromising the initial stiffness. Increasing the value of \( \delta/\delta_n \) up to 5 resulted in a linear degradation for specimen I-0 and a smoother multilinear curve for specimen II-1. When the value of \( \delta/\delta_n \) was no more than 2, a stiffer curve was obtained from specimen II-1 than that of specimen I-0. Then, increasing the ratio from 2 to 5 made a stiffer curve for specimen I-0 as shown in Figure 15(a). This indicates the great performance of energy-dissipating plates on mitigating the stiffness degradation. A greater \( \delta/\delta_n > 5 \) developed splitting cracks for specimen II-1 resulting in a sudden stiffness loss.

Figure 15(b) clearly indicates the impacts of plate thickness on the stiffness degradation from the initial stage. For a small ratio \( \delta/\delta_n < 3 \), the greatest slope of stiffness degradation was found from specimen III-1 \( (t_1 = 8 \text{ mm}) \), followed by specimen II-1 \( (t_1 = 6 \text{ mm}) \) and then specimen II-2 \( (t_1 = 6 \text{ mm}) \). The greatest slope was then obtained from specimen II-1 \( (t_1 = 4 \text{ mm}) \), followed by specimen II-2 \( (t_1 = 6 \text{ mm}) \) and then specimen III-1 \( (t_1 = 8 \text{ mm}) \) when a larger ratio \( \delta/\delta_n > 3 \) was applied. This observation suggests that a slower degradation can be obtained from a thicker plate. At the ultimate stage \( \delta/\delta_n > 5 \), cracking at the end of specimens resulted in stiffer slopes for specimens producing the fastest degradation for specimen II-1 \( (t_1 = 4 \text{ mm}) \) followed by specimen II-2 \( (t_1 = 6 \text{ mm}) \) and then specimen III-1 \( (t_1 = 8 \text{ mm}) \).

Figure 15(c) demonstrates the trend of stiffness degradation resulting from various ratios of axial compression. Increasing the ratio from 0.1 to 0.25 produced three distinguished curves of degrading stiffness. For a value of \( \delta/\delta_n < 5 \), specimen III-2 having a ratio of 0.1 delivered a linear degradation of stiffness. A slower rate of degradation was found for specimens III-1 \( (n = 0.2) \) and III-3 \( (n = 0.25) \). Increasing \( \delta/\delta_n \) up to 3 resulted in a smoother slope of specimen III-2 \( (n = 0.1) \) than that of specimens III-1 \( (n = 0.2) \) and III-3 \( (n = 0.25) \). A further increasing \( \delta/\delta_n \) from 3 to 5 produced a faster degradation for specimen III-2 \( (n = 0.1) \) followed by specimen III-3 \( (n = 0.25) \) and then specimen III-1 \( (n = 0.2) \). The observation suggests limited impacts of axial compression ratio on the stiffness degradation. Eventually, a greater \( \delta/\delta_n > 5 \) failed specimen III-2 \( (n = 0.1) \) and III-3 \( (n = 0.25) \), and split off the end of specimen III-1 \( (n = 0.2) \) degrading the corresponding stiffness.
3.6. Strength Degradation. The factor of $\lambda_j$ [33] was applied to demonstrate the strength degradation at a given load of $j$, which can be obtained from equation (3).

$$
\lambda_j = \frac{V'_j}{V'_j^*},
$$

(3)

$V'_j$ was the maximum load from the $j^{th}$ loading process of the $i^{th}$ cycle.

As shown in Figure 16(a), a value of $\delta/\delta_y < 3$ produced little strength degradation for both control specimen I-0 and specimen II-1 in the usage of energy-dissipating plates, suggesting limited impacts of energy-dissipating plates on improving the strength of this stage. Notable degradation started from the stage of $\delta/\delta_y = 4$, in which specimen II-1 delivered greater degradation than specimen I-0. The trend was even more obvious at $\delta/\delta_y = 5$ while the scale factor aggregated the stress concentrated around the holes and the welding weakness resulting in even quicker strength loss.

Figure 16(b) shows the impacts of plate thickness on strength degradation. Insignificant degradation was found for all selected specimens while a small value of $\delta/\delta_y < 4$ was applied. Notable degradation started from $\delta/\delta_y = 5$. At this stage, the mean value of the absolute factor $\Delta \lambda_j$ was applied to demonstrate the degradation trend. A thinner plate was found to produce greater strength degradation as evidenced by $\Delta \lambda_j$ III-1 > $\Delta \lambda_j$ II-1 > $\Delta \lambda_j$ II-2 > $\Delta \lambda_j$ III-1.

Figure 16(c) indicates the impacts of axial compression ratio, which developed a little difference for $\delta/\delta_y < 3$. This suggests little impact of axial compression ratio at this stage. For $\delta/\delta_y = 4$, a notable degradation was first found from specimen III-2 ($n = 0.1$). The strength degradation of specimens III-2 ($n = 0.1$) and III-3 ($n = 0.25$) is even more obvious while severe cracking of the end degraded the corresponding stiffness at $\delta/\delta_y = 5$. Under a positive load, a greater degradation was obtained from specimen III-1 ($n = 0.2$) than that of specimen III-3 ($n = 0.25$). A negative loading produced more notable strength degradation for specimen III-2 ($n = 0.1$) than that of specimen III-1 ($n = 0.2$). This observation clearly suggests the impacts of axial compression ratio, i.e., a smaller ratio tends to produce a greater strength degradation.

3.7. Energy Dissipation Capacity. The capacity of energy dissipation was determined by the envelope of hysteretic energy curves. A wider and fuller envelope suggests a larger enclosed area resulting in a greater capacity for energy dissipation.

Figure 17(a) shows the relation between cumulative hysteretic energy $E$ [33] and $\delta/\delta_y$ for specimens I-0 and II-1. For a value of $\delta/\delta_y < 5$, increasing $\delta/\delta_y$ produced stiffer curves of cumulative hysteretic energy, suggesting a proportion relation between cumulative hysteretic energy and $\delta/\delta_y$. For a value of $\delta/\delta_y = 5$, specimen II-1 tended to develop smoother curves which resulted from the end-splitting induced stiffness degradation. A greater value of $\delta/\delta_y = 6$ eventually failed specimen II-1 after the first cycle. It should be noted that specimen I-0 performed even better than specimen II-1 in dissipating energy for $\delta/\delta_y > 2$. The reason is the opening crack of specimen II-1 compromised the capacity of energy dissipation.

Figure 17(b) shows the impacts of plate thickness. For a value of $\delta/\delta_y < 3$, stiffer curves were made by increasing the value of $\delta/\delta_y$ before the ultimate. This suggests all specimens performed well to dissipate the energy at this stage. The curves tended to be smoother when a value of $\delta/\delta_y = 5$ was reached. At this stage, the capacity of energy dissipation was gradually reduced, and tests were approaching the ultimate because of a splitting-off failure at the first cycle of the sixth loading process. For a small value of $\delta/\delta_y < 5$, little difference was observed between specimen II-1 ($t_1 = 4$ mm) and II-2 ($t_1 = 6$ mm) which produced curves. At this stage, specimen III-1 ($t_1 = 8$ mm) performed the best in terms of dissipating energy. Those observations suggest the impacts of plate thickness, i.e., a thinner plate tends to result in a greater capacity of energy dissipation.

The impacts of axial compression ratio are shown in Figure 17(c). Stiffer curves of cumulative hysteretic energy were observed as increasing the value of $\delta/\delta_y$ up to 5, indicating a proportion relation between $\delta/\delta_y$ and cumulative energy of dissipation. A value of $\delta/\delta_y = 5$ started to produce a smoother curve for specimen III-1 ($n = 0.2$) which suggested a compromised capacity of energy dissipation because of the side plates cracking. For a value of $\delta/\delta_y > 2$, more energy was dissipated by specimen III-2 ($n = 0.1$), then specimen III-1 ($n = 0.2$) and then III-3 ($n = 0.25$) until specimen III-2 ($n = 0.1$) and III-3 ($n = 0.25$) reached the ultimate failure at $\delta/\delta_y = 5$. This indicates the merits of a smaller ratio of axial compression on dissipating energy.

4. Design Recommendations

4.1. Simplified Equations for Ductility Evaluation. Experimental results clearly show the impacts of plate thickness and axial compression ratio on the ductility of box-shaped steel piers. In this study, a software named 1st Opt was applied to evaluate the simplified equation obtained from regression analyzing the experimental-based ductility coefficient for both loading directions. The simplified equation is listed below:

$$
\mu_i = -1.7570 \times 10^{-23} n^{-22.454} + 2.697 \times 10^{-13} t_i^{13.584} + 5.009, \quad (4)
$$

where $\mu_i$ is ductility coefficient; $n$ is axial compression ratio; $t_i$ is the thickness of energy-dissipating plates.

Table 5 lists computational results from various $\mu_i$, $n$, $t_i$. Manufacture-induced deficiency aggregated the stress concentrated at the end of side plates, producing a larger error of 7.77%.

4.2. Simplified Equations for Load-Bearing Capacity. Based on experimental results, the load-bearing equations for design optimization were delivered, which is listed as below:

$$
C_1 \left( \frac{N}{N_E} \right) + C_2 \left( \frac{M}{M_P} \right) = 1, \quad (5)
$$
where $C_1 = -127 + 302.44\tau; C_2 = 1.263 + 0.0062t_1$; $N$ is the axial maximum compression; $N_\text{el} = \pi^2 E A / \lambda^2$ is the Euler’s critical force; $A$ is the cross-sectional area of box-shaped piers; $\lambda$ is the slenderness ratio; $M$ is the maximum moment; $M_p$ is the plastic moment, $M_p = f_p W$; $W$ is the section modulus.

All computational results are listed in Table 6, in which $M_1$ and $M_2$ are the maximum moments obtained from experimental results and numerical calculations, respectively. It should be noted that $M_2$ agrees well with $M_1$ producing no more than 1% errors for all comparisons. This suggests the merits of the proposed equation on predicting maximum moments.

Figure 18 shows the design procedures in which equation (5) is applied for the load-bearing capacity. Then, equation (4) can be used to secure sufficient ductility for resisting seismic loads and achieving desired safety.

5. Conclusion

Quasi-static experiments have been conducted to explore the impacts of energy-dissipating plates, plate thickness, and axial compression ratio on the seismic behavior of six box-shaped steel piers. Observation and conclusions have been made as given below:

(1) Three failure modes have been observed. The first one was a ductile failure because of the side plate buckling with limited crack openings at the corner of the end. The second failure came from a severe crack splitting off the side plate at the end corner after the side plate buckling. For the third mode, both side plates and energy-dissipating plates were buckling and then severe cracks split the end off.

(2) Energy-dissipating plates worked well to (a) Improve the ductility, deformation, and energy-dissipating capacity, (b) Mitigate the degradation of both stiffness and strength, and (c) Enhance the ultimate load-bearing capacity. Future work is suggested to mitigate the stress concentration around the hole area, which is able to further the merits of the energy-dissipating plate.

(3) A greater ratio of axial compression resulted in a larger capacity in terms of both load bearing and energy dissipation. Increasing the ratio tended to produce less residual deformation suggesting better resilience. The axial compression ratio has limited impacts on strength and stiffness degradation.

(4) Reducing the plate thickness tended to improve the ductility and energy-dissipating capacity, but compromise the load-bearing capacity, and have limited impacts on strength degradation.

(5) Energy-dissipating plates worked well to delay the buckling and cracking of side plates, suggesting a better resilient behavior. A thinner energy-dissipating plate than the side one is able to introduce desired plastic deformation which can be quickly rehabilitated by replacing the fuse element.

(6) In order to facilitate the popularization and application of the new-type steel piers, formulas were established to calculate the bearing capacity and displacement ductility factor of the new-type of box-shaped steel piers.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

This study was supported by Middle-aged Teachers in Education and Science Research of Fujian Province (No. JAT200825), Science Research Project in Xiamen Institute of Technology (Nos. KYT2020001 and KYT2021010), and National Science Foundation of China (No. 51778248).

References


Advances in Materials Science and Engineering