Flexural Behavior of Reinforced Concrete Beams Retrofitted with Modularized Steel Plates

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Abstract: Many structural retrofitting methods tend to only focus on how to improve the strength and ductility of structural members. It is necessary for developing retrofitting strategy to consider not only upgrading the capacity but also achieving rapid and economical construction. In this paper, a new retrofitting details and technique is proposed to improve structural capacity and constructability for retrofitting reinforced concrete beams. The components of retrofitting are prefabricated, and the components are quickly assembled using bolts and chemical anchors on site. The details of modularized steel plates for retrofitting have been chosen based on the finite element analysis. To evaluate the structural performance of concrete beams retrofitted with the proposed details, five concrete beams with and without retrofitting were tested. The proposed retrofitting method significantly increased both the maximum load capacity and ductility of reinforced concrete beams. The test results showed that the flexural performance of the existing reinforced concrete beams increased by 3 times, the ductility by 2.5 times, and the energy dissipation capacity by 7 times.

Keywords: retrofitting; modular; finite element analysis; flexural behavior; ductility

1. Introduction

Structural members need to be strengthened through retrofitting due to revision in the current design code or change in load. The retrofitting methods using jacketing have mainly been investigated by many scholars to improve the strength and ductility of existing structural members. Steel jacketing or fiber-reinforced polymer (FRP) jacketing is a commonly employed method used to retrofit concrete beams and columns.

Steel jacketing consists of steel angles or plates with different thicknesses, widths, and spacing and they are installed by welding. Cement or epoxy mortar fills the gap between the jacket and concrete column. Several studies have been conducted evaluating the structural performance of steel jacketing. Garzon-Roca et al. [1] experimentally investigated the behavior of beam-column joint strengthened with steel caging under combined bending and axial load. The test results showed that steel caging increases both the failure load and ductility of the columns. Adam et al. [2] presented parametric study using the finite element model and carried out analyzing the behavior of reinforced columns strengthened by steel caging. Wei and Wu [3] tested concrete columns retrofitted with high strength steel wire and proposed stress-strain relationship. Tarabia and Albakry [4] conducted an axially loaded test on ten column specimens. The size of steel angles, strip spacing, and grout material were considered as parameters. They reported that most of the specimens were failed due to buckling of the steel angle by crushing of the columns. Alvarez et al. [5] presented the nonlinear analysis procedures for concrete columns retrofitted with steel jackets. Backbone curves were constructed of circular and rectangular retrofitted columns to evaluate the response of jacketing columns. Chrysanidis and Tegos [6] proposed new type of hybrid jackets composed of metal grid jacket and high strength mortar. It has been proven that the proposed systems can improve strength and ductility than conventional method.
This technique is widely used in the construction field due to cost-effective, developing lateral strength, and axial load carrying capacity. However, it is mainly applied to the column due to weight and welding. Additional lifting equipment is required for installation of tube type steel jacket without welding on-site because steel jacket is prefabricated considering member size.

Composite material was used in the jacketing method to improve movement and installation problems. Commonly carbon or aramid fibers with resin or epoxy resin is used for FRP jacketing. Maaddawy [7] examined the effect of eccentrically load of the concrete column retrofitted with FRP jacketing. It was concluded that the strength of columns decreased as eccentricity ratio increased. A number of studies have been evaluated the efficiency of a FRP jacket joining [8–10]. The behavior of concrete columns retrofitted with FRP jacket subjected to seismic loads was investigated based on experimental and analytical results [11,12]. Experimental investigation was performed to explore the performance of eccentrically loaded rectangular RC columns with different CFRP strengthening schemes and preloading levers by Wang et al. [13]. The results indicated that ultimate capacity can be significantly enhanced by full wrapping of CFRP. They also proposed a calculation model to predict the axial load and bending moment capacity of RC columns rapped with FRP jackets.

FRP has low tensile strength and poor adhesion to concrete while FRP sheet have several advantages such as ease of installation and being lightweight. Especially, structural retrofitting methods mainly focus on columns or beam-to-column connections, but only retrofitting vertical members may cause stiffness differences between the horizontal members. Few studies have dealt with retrofitting method for concrete beams than columns or beam-column joints.

In this paper, a new modularized retrofitting method for concrete beams is proposed and experimentally examined. The proposed retrofitting method is expected to resolve the problems in terms of the weight and welding of steel jacketing and adhesion and low strength of FRP jacketing. In order to reduce the self-weight of retrofitted concrete beams, modularize steel plate consisting of L- and Z-shaped steel plates are used instead of steel tube or steel jacket. The new modularized retrofitting method can offer rapid construction progress and good quality. The finite element modeling and parametric study were performed to optimize and determine the details of modularized steel plate to reduce stress concentration and exercise its full capacity. A total of four retrofitted concrete beams and one control beam specimen were fabricated and tested to prove that the proposed retrofitting details were capable of improving the load carrying capacity and ductility.

2. The Retrofitting Method for Reinforced Concrete Beams

The proposed retrofitting system aims at improving structural performance, reducing self-weight and rapid assembly based on modularization. This study proposes a new type of retrofitting details using modularized steel plates. The proposed retrofitting details consist of Z-shaped side plates, L-shaped lower plates, and bottom plates with vertical grid as shown in Figure 1a. Two L-shaped lower plates are fixed to the bottom of the concrete beam using chemical anchors. L-shaped lower plates help easy and accurate installation of the Z-shaped side plates. The Z-shaped side plate is connected to the L-shaped lower plate using bolts and then bonded to the side of the concrete beam using the chemical anchors. The vertical grid is inserted into the Z-shaped side plate and combined using bolts. The space formed under the concrete beam is filled with mortar. The details of concrete beam retrofitted with modularized steel plates are shown in Figure 1b. By assembling L-shaped lower and Z-shaped side plates with bolts, the depth of the parts being extended can be adjusted, and it is easy to design and construct regardless of the size of the members. In addition, grid is installed vertically to resist tensile stress, so no subsequent process is required to install additional flexural reinforcement on the site.
Finite element analysis was performed to determine the details of the proposed retrofitting systems. The commercial FE analysis software ANSYS 16.0 was employed to analyze stress for beams. A three-dimensional and eight-node solid element was applied to the model. The steel plates are fixed to the concrete surface using bolts. Automatic surface to surface contact option was used to model the contact interfaces between concrete and steel plates. A surface element was placed and attached to the beam inside the steel plates and around the steel plates. The thickness, stiffness, and strength were not considered for the surface element. The surface of concrete and steel plates are designated as master and slave surface, respectively. Contact interaction was defined between the concrete and steel plates. The friction contact between the parts was defined using Coulomb friction with friction coefficient of 0.3 [14].

The steel plates and concrete were modelled as isotropic materials. Material and geometric nonlinearity are considered. The elastic-perfectly plastic uniaxial material model with bilinear isotropic hardening is used for the concrete model. The yielding stress is equal to the concrete compressive strength and tangent modulus equals zero. The steel plates and bolts are considered to be elastic-perfectly plastic. Steel are assumed to be linear elastic until the yield stress, and after that, the stress remains constant. The yield strength and ultimate strength of the steel plates were 250 MPa, and 460 MPa, respectively. The compressive strength of the concrete was 24 MPa. The material properties are listed in Table 1.

Table 1. Material properties used in the analytical models.

| Material         | Compressive Strength (MPa) | Modulus of Elasticity (MPa) | Poisson’s Ratio |
|------------------|---------------------------|----------------------------|----------------|
| Concrete         | 24                        | 24,500                     | 0.18           |
| Steel plate      | 250                       | 460                        | 0.3            |

The thickness of the bottom plate (A), the thickness of the L-shaped lower plate (B), the thickness of the Z-shaped side plate (C), and the spacing of bolts (D) were considered as variables as shown in Figure 2. T-shaped beam is designed with a 1000 mm × 150 mm slab, 300 mm × 650 mm beam, and 4 m length. The concrete beam model had simply supported...
condition. The displacement was applied at the node of top surface center of beam in the downward direction. The purpose of the analysis is to compare the stress distribution according to each component details, the displacement of 10 mm was applied for efficiency of analysis. The element mesh size was determined to be 1.5 mm $\times$ 1.5 mm considering that the smallest dimension among the components, L-shaped lower plate and Z-shaped side plate, was 2.5 mm. The finite element model is shown in Figure 3.

![Finite element model](image)

**Figure 3.** Finite element model.

Four models with different mesh sizes, 20 mm, 10 mm, 5 mm, 1.5 mm, were performed to determine a reasonable mesh size that could be reasonably evaluated for the stress of each component. As a results, the stress converged from the mesh size of 5 mm as shown in Figure 4. A sufficient number of elements are needed to observe the stress distribution of each component. Therefore, the mesh size of 1.5 mm would be applicable to this study.

### 3.2. Determinations of the Details of the Proposed Retrofitting System

The bottom plate is located at the bottom of the beam and is the component that occurs high tensile stresses. An appropriate thickness of the bottom plate must be determined. As the thickness of the bottom plate increases, the maximum load capacity increases, which may cause compression failure of the concrete beams. Thickness of the bottom plates are 5 mm, 10 mm, 15 mm, 20 mm, and 30 mm to propose a thickness with a stable stress distribution for the analytical model. The results of the analysis are summarized in Table 2.
Table 2. The maximum stress of components.

| (1) Bottom Plate       | Thickness (mm) | Concrete (MPa) | Side plate (MPa) | Lower plate (MPa) | Bottom plate (MPa) |
|------------------------|----------------|----------------|------------------|-------------------|-------------------|
|                        | 5              | 4.1            | 14.9             | 1.7               | 5.1               |
|                        | 10             | 4.0            | 15.7             | 1.8               | 2.8               |
|                        | 15             | 4.0            | 17.3             | 2.1               | 2.9               |
|                        | 20             | 5.3            | 17.7             | 2.1               | 2.9               |
|                        | 30             | 6.8            | 18.4             | 2.8               | 2.4               |

| (2) L-shaped lower plate | Thickness (mm) | Concrete (MPa) | Side plate (MPa) | Lower plate (MPa) | Bottom plate (MPa) |
|-------------------------|----------------|----------------|------------------|-------------------|-------------------|
|                        | 2.5            | 4.1            | 15.7             | 3.46              | 2.05              |
|                        | 5              | 4.1            | 15.7             | 1.84              | 2.03              |
|                        | 7.5            | 4.1            | 15.6             | 4.3               | 2.03              |
|                        | 10             | 4.0            | 15.6             | 2.0               | 2.02              |

| (3) Z-shped side plate | Thickness (mm) | Concrete (MPa) | Side plate (MPa) | Lower plate (MPa) | Bottom plate (MPa) |
|-----------------------|----------------|----------------|------------------|-------------------|-------------------|
|                       | 2.5            | 3.0            | 11.6             | 2.9               | 1.7               |
|                       | 5              | 4.1            | 15.7             | 1.8               | 2.0               |
|                       | 7.5            | 4.4            | 11.3             | 2.3               | 2.0               |
|                       | 10             | 5.1            | 9.4              | 2.2               | 2.1               |

| (4) Bolt spacing      | Spacing (mm)   | Concrete (MPa) | Side plate (MPa) | Lower plate (MPa) | Bottom plate (MPa) |
|-----------------------|----------------|----------------|------------------|-------------------|-------------------|
|                       | 150            | 4.1            | 15.7             | 2.5               | 1.7               |
|                       | 200            | 4.1            | 15.3             | 1.8               | 2.1               |
|                       | 250            | 4.0            | 16.7             | 5.0               | 2.5               |
|                       | 300            | 3.4            | 16.8             | 6.6               | 2.8               |
|                       | 400            | 2.8            | 20.7             | 6.0               | 2.8               |

The deflection of the beam decreased as the thickness of the bottom plate increased. As the thickness of the bottom plate increased, the stress of the compressive zone and bolt increased. The thickness of the bottom plate is greater than 10 mm, the stress was concentrated on the bolt hole. This may lead to the crushing of concrete before the flexural failure. The thickness of the L-shaped lower plate was alternatively set to 2.5 mm, 5 mm, 7.5 mm, and 10 mm. As the thickness of the L-shaped lower plate increased, the stress on concrete, Z-shaped side plates, and bottom plates decreased slightly, but did not significantly affect the stress distribution. As Z-shaped side plate thickness increases, the maximum stress is occurred in the compressive zone of the concrete beam. It is concluded that the thickness of the Z-shaped side plate can induce the brittle failure on the compressive zone of the con-
crete beam. Bolt spacing should be examined in terms of the composite behavior between the concrete and the modular steel plates. As the bolt spacing increased, the stress of the concrete decreased and the stress of the bolt hole increased. The optimal bolt spacing was determined to be 200 mm, which provided a relatively uniform stress distribution.

![Figure 4](image-url)  
**Figure 4.** Meshing convergence study.

### 4. Experimental Program

#### 4.1. Sequence of Construction

The specimens were manufactured in the order of steel rebar placement and formwork installation, concrete placement, combination of modularize steel plate, and mortar injection. As shown in Figure 5, L-shaped lower plates were fixed to the bottom of the concrete specimen, the process is to bolt the lower and side plates and then insert the bottom plates to tighten the bottom plate and the side plate bolts. After integrating the side plates with chemical anchors, the specimen was manufactured by injecting mortar, and the specimen was manufactured upside down to prevent filling defects in mortar.

![Figure 5](image-url)  
**Figure 5.** The process of proposed retrofitting system application: (a) The installation of the L-shaped lower plate; (b) Assembly of Z-shaped side plate and vertical grid; (c) The process of connecting the side plate used in chemical anchors; (d) The process of grouting of mortar.
4.2. Specimen Details and Test Parameters

In this paper, the total of five specimens were manufactured, including one reinforced concrete beam and four reinforced concrete beams retrofitted with the modularized steel plates. The finite element analysis to optimize the dimensions of each component was performed assuming the real in-situ situations that beams are supporting the slabs. In this study, the slab (flange) parts in T-shaped section only provide the spaces to fix the steel plates by using chemical bolts. Therefore, flanges were not considered when the specimens were manufactured as shown Figure 6. The concrete beams of all specimens were designed to be 300 mm wide, 350 mm high and 4500 mm long. The thickness of Z-shaped side plate was 2.5 mm, the thickness of the L-shaped lower plates was 5 mm, and the spacing of chemical anchors and bolts was 300 mm, using the same in zigzag format. The thickness of the bottom plate, and the number of vertical grids were considered as experimental variables, which are expected to affect the flexural behavior, and represent the corresponding portion of each variable in Figure 5. The depths of the new beams were 100 and 150 mm, the thicknesses of the bottom plates were 5 and 10 mm, and the number of vertical grid were 0, 2, and 4. For the specimen with two vertical grids, the spacing of each grid was 200 mm, and for the specimen with four vertical grids, each grid was installed at spacing of 65 mm. The specimen illustration is organized in Table 3, and the details of the specimen, according to each variable, are shown in Figure 7. The 28-day compressive strength of the concrete used in this experiment was 24 MPa. For the tensile and compressive reinforcement, deformed rebar was used with a diameter of 19 mm and it’s yield strength is 400 MPa. Steel plate was used with a yield strength of 275 MPa. The test results of compressive strength of concrete and steel plate are shown in Figure 8. The high-strength bolts which is F10T M16 bolts with a diameter of 16 mm was used to connect the concrete beams. The material properties are summarized in Table 4.
Table 3. The test specimen parameters.

| No. | Name   | Added Beam Depth (mm) | Bottom Plate Thickness (mm) | Vertical Grid (EA) |
|-----|--------|------------------------|----------------------------|--------------------|
| 1   | RC     | -                      | -                          | -                  |
| 2   | D100-A | 100                    | 5                          | 2                  |
| 3   | D100-B2 | 100                    | 10                         | 2                  |
| 4   | D100-B4 | 100                    | 10                         | 4                  |
| 5   | D100-B0 | 100                    | 10                         | -                  |

1: Bottom plate thickness (A: 5 mm, B: 10 mm). 2: The number of steel plate openings (2: 2EA, 4: 4EA, 0: 0EA).

4.3. Test Set Up

All specimens were tested using a Universal Testing Machine (UTM) with a 5000 kN load capacity as shown in Figure 9. The specimens were simply supported and subjected to four-point bending loading. The distance from support to loading point was 1750 mm and the distance between the two central point loads was 600 mm. Displacement was applied to each specimen a rate of 2 mm/min. During the flexural test, load and midspan deflection were measured. Deflection was measured with three linear variable displacement transducers (LVDT’s) mounted onto the bottom of the specimens. For concrete, the strain gage is attached in the compression zone C1. In order to evaluate the yield by strain of the tensile reinforcing bar, the strain gage was attached in the middle of the tensile reinforcing bar.
bar before concrete placement (R1). The strain gage (R2) was attached to the center of the vertical grid’s vertical strip to evaluate the flexural behavior as tensile reinforcement. A total of 7 strain gages (S1 ~ S7) were attached to the front of the beam to evaluate the local buckling and deformation of the modularized steel plates. The installation of the LVDTs and strain gages is shown in Figure 10. All specimens were continuously observed to mark the crack pattern and note any signs of failure during the loading process. Loading, displacement and strain were recorded using a data acquisition system. The specimens were tested until the load decreased at the level of approximately 70% to 80% of the maximum load.

![Stress-Strain Curve](image1)

![Stress-Strain Curve](image2)

Figure 8. (a) Compressive strength test of concrete; (b) Tensile test of steel plate.
Table 4. Material properties.

| Material | Compressive Strength (MPa) |
|----------|-----------------------------|
| Concrete | 24                          |

| Material     | Diameter (mm) | Yield Strength (MPa) | Modulus of Elasticity (MPa) |
|--------------|---------------|----------------------|-----------------------------|
| Rebar        | 19            | 400                  | 200,000                     |
| Stirrup      | 10            | 400                  | 200,000                     |

| Material     | Yield Strength (MPa) | Tensile Strength (MPa) | Modulus of Elasticity (MPa) |
|--------------|----------------------|------------------------|----------------------------|
| Steel plate  | 275                  | 410–550                | 205,000                    |

Figure 9. Test setup: (a) A schematic of the test setup; (b) A photograph of the test setup.

Figure 10. The locations of strain gauges.
5. Test Results and Discussion
5.1. Cracking Behavior and Modes of Failure

Table 5 shows the crack pattern at maximum load for each specimen tested. For the D100-A2 and D100-B2 specimens with variable thicknesses of the bottom plate, it has been confirmed that the crushing of compression zone widens as the thickness of the bottom plate increases. As the number of vertical grid increased, the crushing and spalling of compression zone were more localized.

| Name            | Crack Propagation                                                                 | Observed Damage                                                                 |
|-----------------|-----------------------------------------------------------------------------------|----------------------------------------------------------------------------------|
| Bottom plate thickness | ![D100-A2](image1)                                                                 | -Concrete crushing at the top of the concrete                                   |
|                 | ![D100-B2](image2)                                                                | -As the bottom plate thickness increases, cracks propagate widely                |
|                 | ![D100-B0](image3)                                                                |                                                                                 |
| The number of vertical grid | ![D100-B2](image4)                                                                 | -Concrete crushing at the top of the concrete                                   |
|                 | ![D100-B4](image5)                                                                | -As the number of steel plates with openings increases, the concrete crushing occurs more locally. |

The loading was terminated when the load was reduced 80% after the maximum load was reached. All the specimens showed flexural behavior as shown in Figure 11, cracks started appearing in the compression zone. As the load increased, the cracks of the compression zone became wider, and some spalling was observed on the mortar grouted on the bottom plate. The spalling of the mortar was observed differently according to the vertical grid. More spalling occurred in the D100-B0 specimen without the vertical grid. Finally, as the concrete crushing and spalling increased, the load gradually decreased. Local buckling of the Z-shape side plates and bolt fracture were not observed in all retrofitted specimens as shown in Figures 12 and 13. It is concluded that sliding was not observed at the end of the beam because the steel plates and concrete were perfectly bonded.
The test results are summarized in Table 6. The nominal flexural strengths of the specimens are 153.2 kN for D100-A2, 172.6 kN for D100-B0, 176.2 kN for D100-B2, and 175.1 kN for D100-B4. These results indicate that the flexural strength increased as the thickness of the bottom plate increased.

As the bottom plate thickness increases, the concrete crushing at the top gate became more localized. The concrete spalling was observed differently according to the presence of vertical grid. This is to achieve the restraining effect of reinforcing parts on the mortar injected into reinforcing areas, and the flexural strength due to reinforcement is improved depending on whether or not it is installed.

Figure 11. The shape of the test specimen after a flexural test.

Figure 12. The side plate and bottom plate after a flexural test.

Figure 13. Load-Deflection Relationship.

The Load-displacement curve is shown in Figure 14. Specimens retrofitted with modularized steel plates showed greater stiffness and maximum load than RC specimen. Retrofitted specimens showed similar initial stiffness. Remarkable differences were not observed in terms of the number of vertical grid between D100-B2 and D100-B4. The maximum load for D100-B2 was 602 kN and that for D100-B4 was 598 kN, but the maximum load for D100-B0 was 514 kN, which was increased by approximately 1.17 times depending on the presence of vertical grid. This is to achieve the restraining effect of reinforcing parts on the mortar injected into reinforcing areas, and the flexural strength due to reinforcement is improved depending on whether or not it is installed.
The maximum load increased as the thickness of bottom plate increased. A comparison of the load-displacement curves of the two specimens confirmed that the maximum load for the D100-A2 specimen was 508.48 kN, the maximum load for the D100-B2 specimen was 602.62 kN, and that the flexural performance was approximately 1.19 times higher for the D100-B2 specimen.

The test results are summarized in Table 6. The nominal flexural strength $P_n$ is calculated from plastic stress distribution method suggested by AISC 360-10 [15]. The yield load $P_y$ defines a point as the yield load where a line parallel to the line connecting the origin to the maximum load meets the load-displacement curve as shown in Figure 15. The proposed retrofitting method has shown an increase in the strength, with increasing thicknesses of bottom plates and the number of vertical grids. The experiment resulted in an average increase of 3.2 times the maximum load and an average increase of 1.3 times the maximum displacement.

Table 6. Flexural test results.

| No. | Name      | $P_n$ 1 (kN) | $P_{y,\text{test}}$ 2 (kN) | $P_{u,\text{test}}$ 3 (kN) | $P_u/P_n$ | $P_u/P_y$ | $\delta_y$ 4 (mm) | $\delta_u$ 5 (mm) |
|-----|-----------|-------------|-----------------|-----------------|-----------|-----------|-----------------|-----------------|
| 1   | RC        | 138.06      | 137.55          | 171.65          | 1.24      | 1.08      | 18.09           | 38.72           |
| 2   | D100-A2   | 535.19      | 370.86          | 508.48          | 0.95      | 1.11      | 13.48           | 47.89           |
| 3   | D100-B2   | 585.03      | 466.03          | 602.62          | 1.03      | 1.08      | 16.27           | 52.21           |
| 4   | D100-B4   | 585.03      | 423.40          | 597.55          | 1.02      | 1.07      | 13.90           | 49.25           |
| 5   | D100-B0   | 556.73      | 382.13          | 514.00          | 0.92      | 1.10      | 14.61           | 50.86           |

1 $P_n$: The nominal force calculated by the nominal flexural strength applied to the beam. 2 $P_{y,\text{test}}$: The yield load of each specimen. 3 $P_{u,\text{test}}$: The maximum load of each specimen. 4 $\delta_y$: The displacement when the yield load applied. 5 $\delta_u$: The displacement when the maximum load applied.

5.2. Load-Strain Relationship

Figure 16a shows the load-strain relationship of the C1 gauge located in the upper of the concrete specimen. The strains of all the specimens were similar and after exceeding the maximum compressive strain of the concrete under the maximum load of 0.003, the gauge was found to have been damaged due to the compressions on the top of the concrete. Figure 16b shows the load-strain relationship for the R2 gauge located at the vertical grid. Although there were differences in the slopes of the initial linear depending on the specimen, the strain of the specimens increased in a similar, and the yield strain reached
0.002 under the maximum load and then yielded. It is judged that the specimens exhibited sufficient flexural behavior because vertical grid as tensile reinforcement yielded.

![Figure 15. Determination of the yield strength.](image)

5.3. Ductility Capacity

The ductility of flexural members, such as beams, can be calculated by the ratio of the maximum displacement and yield displacement after the maximum load is generated, without a sudden reduction of the load when the load is reduced to 70–80%. In this study, the ductility was calculated by dividing the displacement $\delta_{Y}$ when the maximum load was reduced to 80% of the maximum load after the maximum load was applied by the displacement $\delta_u$ when the yield load was applied. The ductility capacities of each specimen are summarized, as shown in Table 7. The ductility capacity of specimens retrofitted steel plates was shown to be at least 7.47 and up to 10.01, averaging 8.85. Compared to the RC specimen, the ductility capacity is about 2.5 times higher. The ductility of each D100-A2 and D100-B2 specimens, with variable thicknesses of the bottom plate, was 10.01 and 7.47, respectively, indicating that the ductility decreases as the thickness of the bottom plate increases. In addition, the comparison of specimens according to the number of vertical grid resulted in improved ductility as the number of vertical grid increased to 7.47 for D100-B4, 9.67 for D100-B0, and 8.26 for the D100-B0 specimen. However, due to the thickness of the bottom plate and the presence or absence of vertical grid, excessive reinforcement on the tensile zone is considered to be possible due to the crushing of concrete. The ductility capability can be improved by the application of modularized steel plates.

![Figure 16. The Load-strain relationships: (a) C1 Guage; (b) R2 Gauge.](image)
Table 7. Analysis of the ductility capacity for each specimen.

| No. | Name   | $\delta_{0.8Pu}^1$ | $\delta_y^2$ | $\delta_{0.8Pu}/\delta_y^3$ |
|-----|--------|--------------------|--------------|-----------------------------|
| 1   | RC     | 64.78              | 18.09        | 3.58                        |
| 2   | D100-A2| 134.88             | 13.48        | 10.01                       |
| 3   | D100-B2| 121.52             | 16.27        | 7.47                        |
| 4   | D100-B4| 134.43             | 13.90        | 9.67                        |
| 5   | D100-B0| 120.75             | 14.61        | 8.26                        |

1 $\delta_{0.8Pu}$: The displacement when the maximum load reduced to 80%. 2 $\delta_y$: The displacement when the yield load applied. 3 $\delta_{0.8Pu}/\delta_y$: The ductility capacity of each specimen.

5.4. Energy Dissipation Capacity

Energy dissipation is one of the structural performance evaluation elements of a member that measures the ability of the structure and member to absorb the energy generated when deformation is applied to the structure and components. An analysis of the energy dissipation capacity of specimens is shown in Figure 17.

For the energy dissipation with a thickness increase of the bottom plate, the D100-A2 specimen measured 61.6 kN-m, and the D100-B2 specimen measured 64 kN-m, and the difference with varying thickness of the bottom plate was minimal. The D100-B2 specimen, D100-B4 specimen, and D100-B0 specimen, with variable numbers of vertical grid, are shown to improve their energy dissipation capacity as the number of vertical grid increases to 64 kN-m, 71.6 kN-m, and 55 kN-m, respectively. It has been confirmed that the energy dissipation capability was approximately 7 times higher than that RC specimen.

5.5. Model Validation and Discussion

The results of the numerical model are validated and confirmed by comparing the results of the experiments and the numerical model. Analysis conditions were set the identical as for parametric study. For comparison with the experimental results, a displacement of 120 mm was applied. Figure 18 shows the comparison of the numerical predictions with the experimental results for the D100-B4 specimen. The comparison indicated that numerical results could reasonably predict the overall structural performance in terms of initial stiffness and maximum load capacity. The numerical models slightly became stiffer than the experimental results. This may be due to the numerical model not considering micro-crack or the construction error of the specimen, but it is considered to be acceptable. However, the numerical model shows a different behavior after yielding due to the limita-
tion of the numerical models in implementing the failure mechanism. This is a topic for further investigation by the authors.

![Comparison between the numerical and the experimental results of load versus midspan deflection.](image)

**Figure 18.** Comparison between the numerical and the experimental results of load versus midspan deflection.

### 6. Conclusions

The purpose of this study is to propose and examine a modularized steel plate retrofitting method resulting in improved structural performance and constructability. The details of retrofitting method were proposed based on the finite element analysis. The flexural test was conducted to evaluate the structural performance of concrete beams retrofitted with the proposed method. The main conclusions obtained from the study are as follows:

1. In this study, the details and components of the modularized retrofitting method are proposed, with Z-shaped side plates, L-shaped lower plate, and bottom plate with vertical grid. Finite element analyses were carried out to investigate the effect of the bottom plate thickness, L-shaped lower plate thickness, Z-shaped side plate thickness, and the bolt spacing on the stress distribution. Details of the modularized steel plate retrofitting system were proposed based on the analysis results.

2. The maximum load increased as the thickness of the bottom plate increased, but it was found to be relatively lacking in ductility. Although the number of vertical grid did not significantly affect stiffness, the increase in that number improved the ductility. In addition, the flexural performance increased by about 1.17 times, depending on the presence of the vertical grid. The vertical grid attached to the bottom plate increased the confinement effect of the mortar and reduced spalling by acting as a flexural reinforcement.

3. The ductility and energy dissipation capabilities were compared and analyzed. Compared to non-retrofitted reinforced concrete beams, the ductility capacity increased by about 2.5 times, and the energy dissipation capacity increased by about seven times. This confirms that the application of the proposed retrofitting method can improve the structural performance of existing concrete members.

4. The experiment in this paper focused on the evaluation of the flexural capacity of retrofitted concrete beams. Further experimental research is needed on the shear behavior of concrete beams retrofitted with modularized steel plates. Moreover, a finite element model that can predict structural capacity is required for the design stage.
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