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# THE EFFECT OF JACKET AREA ON THE BEHAVIOR OF REPAIRED/STRENGTHENED REINFORCED CONCRETE COLUMNS

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**Abstract:** Jacketing is the most popular method for strengthening reinforced concrete columns under axial loading. This repair and strengthening technique is widely used not only in Türkiye but also in other technologically developed countries. In this study, the effectiveness of this method is investigated experimentally, by testing two test parameters which are column section geometry and jacket thickness. The specimens were enlarged on all four sides to understand how the thickness of the jacket layer affects the behavior of the column. Using the experimental data, load versus deformation curves were plotted for each test specimen. The parameters (strength, rigidity, ductility, and energy dissipation capacity) that determine the behaviour of the reinforced column were then analyzed. At the end of this study, the effects of column section geometry and jacket thickness on the jacketing method are discussed, providing valuable preliminary data for future experimental studies.

**Keywords:** Strengthening, Repair, Jacketing, Axial load, Reinforced concrete, Column

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## 1. Introduction

Since the repair and strengthening of reinforced concrete columns is a widely used technique, many studies have been found in the literature on the effectiveness of jacketing technique, which is one of the strengthening methods, and the effects of jacketing on behavior and strength. In a master's thesis by Aksan (1988), the effectiveness of different types of jacketing was investigated only in terms of axial load-carrying capacity. For this purpose, a series of experiments consisting of five test specimens were carried out. Four of them reflect various types of jacketing in terms of application conditions, while the fifth is a monolithic reference specimen with the same cross-section, properties and reinforcement as the jacketed columns in order to determine the effectiveness of different jacketing methods. Although 70-80% of the strength of the monolithic specimen was reached with the repair jacketing in the unloaded condition, the repair jacketing in the loaded condition did not exhibit the same performance and this repair jacketing could only reach 50% of the strength of the monolithic specimen and only the jacketing was loaded. The ductility, energy consumption and similarly stiffness of the repaired members are less than the monolithic and reinforced members both under load and unloaded conditions. In addition, excessive deformations were observed in the repaired specimens as a result of reloading.

Suleiman (1991) experimentally investigated the behavior of jacketed or strengthened reinforced concrete columns under axial load and single curvature bending. Three of the five specimens were tested under uniform load or reversible load and then these members were jacketed and retested. According to the jacketing, the repair and strengthening jacketing was named as repair and strengthening jacketing depending on the damage in the lean test specimen. In addition to these specimens, two monolithic reference specimens were also tested. It was shown that the reinforced jacketed specimens behaved as well as the monolithic reference specimen under both monotonic and reversible loads. In the case of the repaired jacketed members, both stiffness and strength were observed to be smaller than the monolithic reference member.

Yanarateş (1990) investigated the repair and strengthening of axially loaded columns under eccentric loading in an experimental study. Repair of columns damaged under axial load by jacketing was carried out under unloaded conditions after the load was removed. It was concluded that the use of Expanding Metal Mesh (EMM) as wrapping reinforcement was not an effective solution for the columns that were retested after repair.

Yumak (1991) was observed that jacketed columns anchored with epoxy and mechanical methods exhibited good behavior. In the case of discontinuous longitudinal reinforcement; the results obtained were not very reassuring. Can (1995) concluded that the stirrup used as wrapping reinforcement during jacketing is highly effective in the success of the jacketing, and if possible, a strong and closed stirrup surrounding the entire column by stripping the rust allowances on the back face of the column will increase the success of the jacketing. Can (1995b) found that 92% of the monolithic section capacity could be achieved with reinforcement jacketing on four sides of the columns and 88% with repair jacketing. However, the columns jacketed on four faces (repaired and strengthened) exhibited a successful behavior in terms of strength, stiffness and ductility in general. In columns jacketed on three sides, 90% of the monolithic section capacity could be reached with the strengthening jacket and 82% with the repair jacket. Although the stiffness and ductility of the repaired and strengthened columns are close to the monolithic column, a certain reduction in energy consumption is observed. The strength of the reinforced column jacketed from two adjacent faces was 89% of the monolithic column and 86% of the repaired column. Although the ductility and energy consumption of the jacketed columns (retrofit and repair) were lower compared to the monolithic column, these specimens exhibited a good behavior in terms of stiffness. In the whole experimental study, the adjacent two-face and three-face jacketed columns exhibited lower values in terms of strength and ductility compared to the four-face jacketed columns.

Demirel et al. (1995) determined that the z-shaped bars connecting the additional longitudinal reinforcement of the jacketing and the existing longitudinal reinforcement, the spacing of these bars, and the way they are welded have a significant effect on the strength and behavior. Cısdık (1998) tested the usability of fretted jacketing for repair and strengthening of columns under axial loads. The research was carried out on two different types of columns with fretted and circular ribs. As a result of the experiments, it was found that in both types of columns, the bare column bearing strength increased by at least 60-80% with jacketing and the initial brittle behavior became ductile. The load-unit deformation curves of the columns were drawn with the data obtained and as a result of the evaluations, it was concluded that fretted jacketing can be safely used in the repair and strengthening of reinforced concrete columns.

After jacketing, strengthening/repair with steel, CFRP and TRM has been intensively studied in the strengthening/repair literature for the last 10 years. Structural specimens were strengthened by jacketing in the nineties and then by bonding steel plates to the tensile and shear surfaces with adhesives and bolts. This method has been used for many years, but it has been abandoned over time due to reasons such as the need for skilled workmanship and changing the architectural appearance of the building, the need to protect the steel specimens against effects such as corrosion and fire, the added steel reinforcement specimens changing the structural dynamics characteristics due to their weight and installation difficulties (Rochdi et al. (2006)). Fiber-reinforced polymer fabrics (FRP), which are produced from composite materials, have become a widely preferred alternative to reinforcement with steel strips due to their high mechanical strength values, easy application due to their light weight, ease of installation, properties that do not change the dynamic characteristics since they do not add additional weight to the structure and high resistance to environmental effects. FRP reinforcement has been the most preferred construction material for strengthening structures for the last 20 years (Ozbakkaloglu et al., 2013; Ghoroubi et al., 2020; Mercimek et al., 2021; Mercimek et al., 2023; Türer et al., 2023). Epoxy material with organic structure was used to form a cohesive zone between the FRP and the surface to which the FRP was applied. However, (a) poor resistance to fire (Triantafillou and Thanasis (2006)), (b) inapplicability on wet or damp surfaces (Bournas and Thanasis, 2007), (c) lack of ability to replace the coating layer (Francisco et al., 2012) (d) classification as a hazardous material during disposal, (e) high cost of FTEK techniques (Francisco et al., 2012), (f) permeability: insufficient vapor permeability and the use of organic resins can cause damage to concrete (Triantafillou and Thanasis (2006)). Considering these negativities, the widespread use of TTH through scientific research has been recognized as a remarkable progress in the field of structural reinforcement (Triantafillou and Thanasis, 2006; Bournas and Thanasis, 2007). TTH is a composite building material consisting of a cement-based inorganic mortar with textiles made of different materials (steel, carbon, basalt, glass, etc.). Textiles used as reinforcement of the composite material typically consist of strands of fibers oriented perpendicular to each other (bi-directional). Considering that the mortar is produced and applied by conventional methods, it was realized that TTH has some advantages over FTEK. These advantages are; (a) low cost, (b) resistance to high temperatures, (c) applicability to concrete, reinforced concrete and masonry surfaces, (d) applicability to wet surfaces, (e) low thermal transmittance and (f) high bearing strength. Considering these advantages, researchers have widely increased the use of TTH in the development of retrofit/repair details in the last 10 years (Mercimek, 2023; Mercimek et al., 2022; Mercimek et al., 2024).

After the February 6, 2023 Kahramanmaraş earthquakes, retrofitting projects were prepared for many buildings in the earthquake zone. It was observed that many engineers who prepared these projects strengthened the moderately damaged structures by jacketing the columns and adding shear walls. Although it is a subject that has been researched since the early nineties, some deficiencies have been identified at many critical points. A thesis study was conducted by Kopraman (2003) on the subject. In this study, based on Kopramans (2003), aimed to investigate the behavior of columns with enlarged cross sections by jacketing method. During the application of this method, a reinforced concrete jacket was formed by placing additional longitudinal reinforcement at all four corners of the existing column and the same wrapping reinforcement as the plain column reinforcement along the column and the column cross section was enlarged in three different ratios. The column cross sections were selected so that these ratios were similar for both rectangular and square columns. For the two different types of column geometries, the characteristic ratios of Acj/Acc=0.8 for the narrow type mantle, Acj/Acc=1.0 for the normal type mantle and Acj/Acc=1.8 for the wide type mantle were tried to be achieved. At the end of the experiment, these ratios were found to be sufficient to give meaningful results in terms of purposeful behavior.

## 2. Materials and Method

The specifications of the experimental specimens are presented in Table 1. Since three different mantle areas were determined, three lean columns were fabricated for each cross-section. However, jacketing parameters were not tested for columns with square cross-sections in the repair group experiments. The main reason for this is that the jacketing method has been applied on square and circular section specimens more than rectangular section columns (Cısdık, 1998; Ünsal 1998). For this reason, jacketing was applied only to rectangular columns for the repair group. In the retrofitting group experiments, square and rectangular columns were tested as a complete set.

In addition to these test specimens, two monolithic (single cast) specimens, one with square and one with rectangular cross-section, with normal type jacketing, were manufactured. The monolithic members were included in the scope of the experiments to provide a reference for the specimens. Fourteen experimental specimens were tested within the scope of these experiments. The schematic view and donate plans of the specimens are shown in Figure 1. Some images of the production stages of the experimental specimens are shared in Figure 2.

Care was taken to ensure that the concrete used in the test specimens was easy to place in the mold and had high strength. In order to repeat these properties in different castings, a mixture ratio that was found suitable for the casting of the experimental specimens was used as a standard. For this reason, different mixtures were made and broken after 28 days of strength. As a result, it was decided to use two mixtures with different workability properties. During the concreting of the specimens, at least seven cylinder samples were taken from each concrete mix. The cylinder specimens were of standard dimensions, 150 mm in diameter and 300 mm in height. The specimens were subjected to the same vibration as the test specimen and placed in the formwork. However, they were kept in the same place and condition as the specimens in order to ensure that the specimen strengths were representative of the specimen strengths. The concrete compressive strengths of the specimens are given in Table 2.

**Table 1.** Experimental groups, names and cross-sectional properties of specimens

| Specimen | | Column Section | | Jacketed Column | | Cross-section geometry | Acj / Acc |
|---|---|---|---|---|---|---|---|
| | | Dimension (mm) | h/b | Dimension (mm) | h/b | | |
| Strengthened Specimens | S1 | 120x120 | 1.00 | 170x170 | 1.00 | Square | 1.01 |
| | S2 | 120x120 | 1.00 | 160x160 | 1.00 | Square | 0.78 |
| | S3 | 120x120 | 1.00 | 200x200 | 1.00 | Square | 1.78 |
| | S4 | 90x160 | 1.78 | 140x210 | 1.54 | Rectangle | 1.04 |
| | S5 | 90x160 | 1.78 | 130x200 | 1.50 | Rectangle | 0.81 |
| | S6 | 90x160 | 1.78 | 170x240 | 1.41 | Rectangle | 1.83 |
| Damaged Specimens | B4 | 90x160 | 1.78 | --- | --- | Rectangle | --- |
| | B5 | 90x160 | 1.78 | --- | --- | Rectangle | --- |
| | B6 | 90x160 | 1.78 | --- | --- | Rectangle | --- |
| Repaired Specimens | R4 | 90x160 | 1.78 | 140x210 | 1.54 | Rectangle | 1.04 |
| | R5 | 90x160 | 1.78 | 130x200 | 1.50 | Rectangle | 0.81 |
| | R6 | 90x160 | 1.78 | 170x240 | 1.41 | Rectangle | 1.83 |
| Reference Specimens | M1 | --- | --- | 170x170 | 1.00 | Square | 1.01 |
| | M4 | --- | --- | 140x210 | 1.50 | Rectangle | 1.04 |

**Table 2.** Compressive strength of concrete material

| Specimen | Column (MPa) | Jacket (MPa) |
|---|---|---|
| S1,S2,S3 | 18.8 | 23.9 |
| S4,S5,S6 | 19.4 | 23.9 |
| R4, R5, R6 | 18.8 | 18.9 |
| M1,M4 | 23.0 | --- |

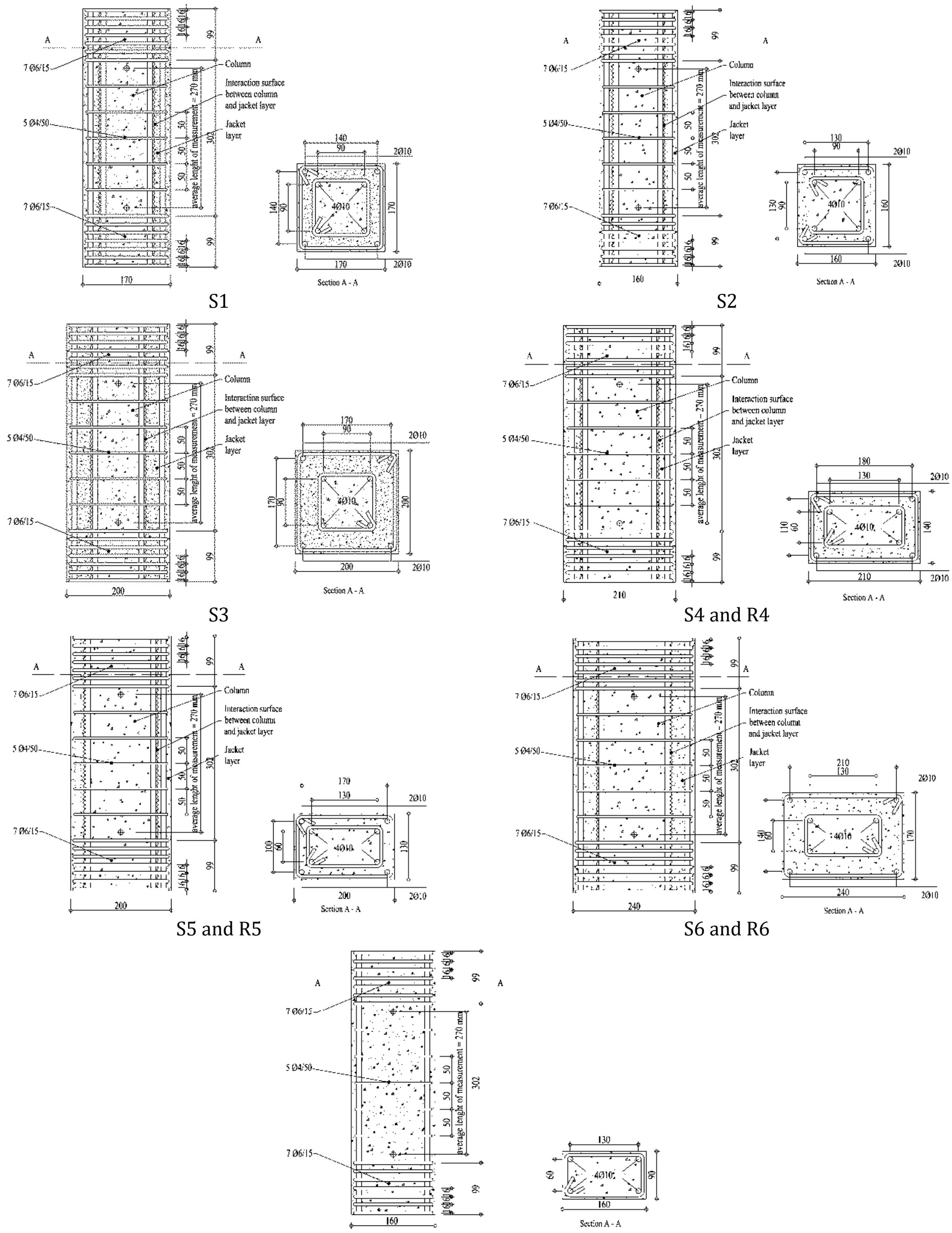



**Figure 1.** Dimensions and reinforcement detail of specimens

Steel reinforcement with flat surface was used in all experimental specimens. In the construction of the experimental specimens, Ø10 was used as longitudinal reinforcement. Ø4 reinforcement was used in the middle region of the column and Ø6 reinforcement was used in the end regions of the column.

The same reinforcements were used in all of the specimens. A sufficient number of samples were taken from these reinforcements and their average yield strengths were determined. The yield strengths of the reinforcements are given in Table 3.

**Table 3.** Yield strength of the reinforcements used in the experiments

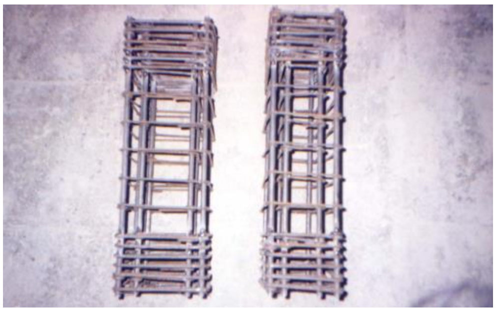
| Diameter (mm) | Yield Strength (MPa) |
|---|---|
| ϕ 4 | 450 |
| ϕ 6 | 362 |
| ϕ 10 | 365 |


(a)

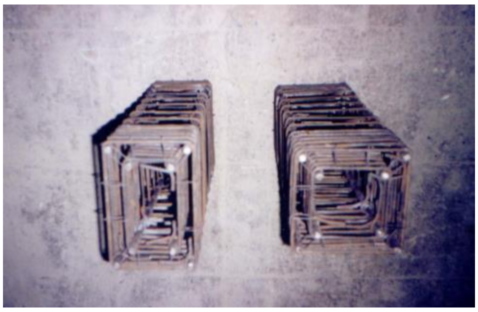

(b)


(c)


(d)



(e)



(f)

**Figure 2.** Production process of specimens

The experimental setup is shown in Figure 3. The loading frame, which is the main specimen of the setup, is 2 m high. During the experiment, a load of 2000 kN was applied to the specimens with a hydraulic jack. The loading was tried to be done at constant speed. The applied load was transmitted to the scanner-reader with a load meter with a pressure capacity of 2000 kN and recorded on the computer. Displacement records were taken from 4 displacement meters over the experimental specimens.

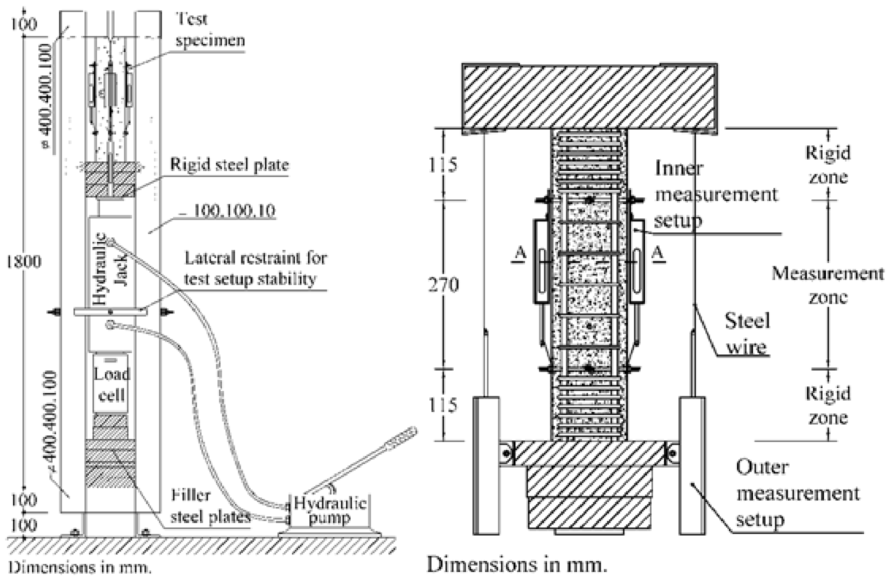



**Figure 3.** Experiment and measurement setup.

## 3. Results

The load-displacement graphs of the specimens are shown in Figure 4. In drawing the load-unit deformation relationship of the specimens, the deformation readings on the four faces were averaged and interpretations were made accordingly. The damage distribution of the specimens is given in Figure 5. Since the concretes of all the test specimens could not be prepared simultaneously, the experiments were carried out on specimens with concretes of different strengths. For this reason, the load-unit deformation relationships of the specimens were first rearranged according to a common concrete strength (fc=20 MPa) and all test results were evaluated based on these curves. In this evaluation, strength, strength degradation, energy dissipation capacity, ductility and stiffness were considered as the determining parameters of the specimen behavior. In order to determine the success of the method, firstly, the reinforced and repaired specimens were compared with a cast (monolithic) specimen.

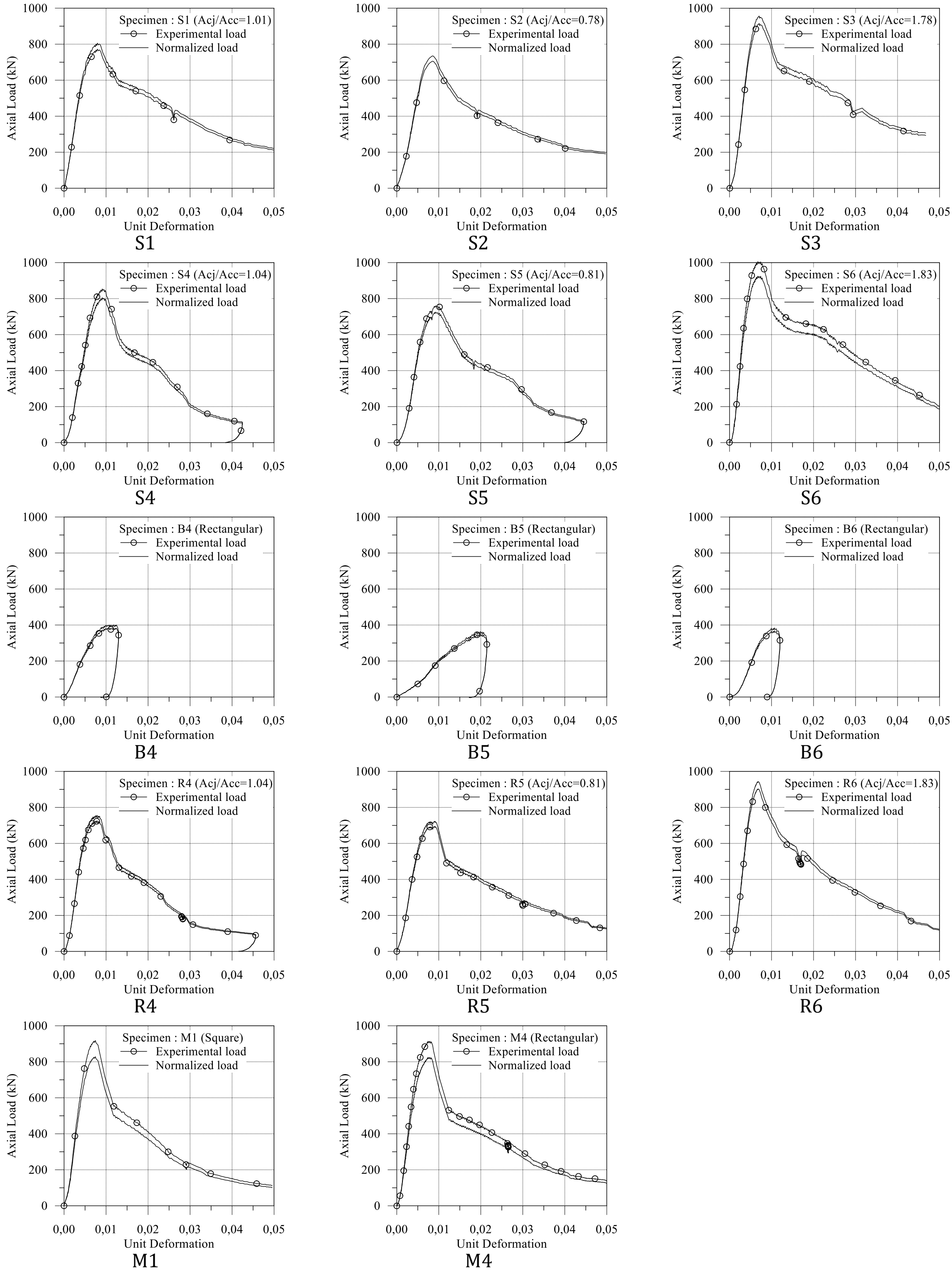



**Figure 4.** Load-displacement graphs


**Figure 5.** Damage distribution of specimens after test

The S1 specimen did not show any damage until the 429 kN axial load level, but after this point, damage effects started to be seen in the member with whitewash blistering starting from the south face. At 630 kN load, 2.83 mm axial deformation occurred in the specimen and crushing was observed in the upper and lower rigid zones outside the measurement zone on the west face. This crushing proceeded directly into the measurement zone of the specimen. When the specimen deformed by 4.31 mm in response to an axial load of 768 kN, a sudden collapse occurred inside the measurement zone.

In the S2 specimen, whitewash blistering at the stirrup line occurred under a deformation of 2.14 mm against 377 kN. Crushing first occurred in the upper rigid zone outside the measurement zone on the east face under an axial force of 594 kN. When 705 kN and 4.19 mm deformation was read, the specimen started to lose strength. Buckling of the longitudinal reinforcement started after the peak point was exceeded and the reinforcement at the south-east corner buckled at 395 kN and then the reinforcement at the south-west corner buckled.

In the S3 test specimen, the swelling at the stirrups started when the axial load was at 585 kN. After the swellings occurred at the stirrups on all faces, crushing started at the corners of the rigid region of the specimen when the load increased to 686 kN. Crushing, which first started outside the south-east measurement zone, was observed outside the upper and lower measurement zone at the north-west corner at 882 kN. After the bearing capacity peak of 919 kN and 3.00 mm was exceeded, the shear plane did not draw its direction precisely within the measurement zone until the deformation point of 802 kN and 3.93 mm, and was observed as local dents. After this point, it was observed that the shear plane made an angle of 600 with the horizontal when viewed from the north, and the crushing zone, which started outside the measurement zone on the west face, descended into the measurement zone on the same face and then proceeded towards the lower boundary of the measurement zone on the east face. It was observed that the reinforcement at the south-west and south-east corners buckled under a displacement of 10.67 mm and an axial load of 553 kN.

Although there was a deformation of 5.09 mm and an axial load of 815 kN in the S4 test specimen, there was no crushing progression within the measurement zone. While the specimen with 5.84 mm deformation under 846 kN load exceeded this peak, the crushes formed in the shell concrete and generally in the lower rigid zone rapidly progressed into the measurement zone. The experiment was terminated with a deformation of 20 mm in the S5 test specimen. No stirrup opening or rupture was observed in the mantle and lean sections. However, the ups and downs seen in the load-deformation curve of the specimen are thought to be caused by the roughness of the loading plane. This uneven plane was flattened with the effect of the load and lost its negative effect when it exceeded the peak point of the specimen. S6 member was subjected to 40 mm axial deformation. The failure was caused by the shear plane starting from the upper region on the north face, moving downwards and making an angle of 45o with the horizontal. The vertical cracks formed during the experiment indicate that concentric loading was applied more successfully in this experiment compared to the other experiments.

Care was taken to avoid excessive damage to the B4 test specimen. The longitudinal reinforcements have ejected the shell concrete. However, none of them reached the buckling level. The experiment was terminated according to the behaviour of the specimen. At the end of the experiment, the total deformation of the member was 5.81 mm for an axial load of 267 kN. In the B5 specimen, the total deformation of the member was recorded as 10.07 mm when it reached the axial load level of 272 kN, which is the bearing capacity. The reason for the excessive axial deformation in this specimen is the early damage to the corners of the rigid zones. For this reason, the axial force value carried by the specimen remained at a low level. Experimental specimen B6 reached a bearing capacity of 288 kN at an axial deformation level of 4.80 mm. The specimen behaved quite well, allowing similar loading effects to be observed on all four faces. However, in this specimen, the behavior of the member was also affected by the early crushing at the top and bottom of the rigid zone. As a result of the observations, no buckling of the longitudinal reinforcement was observed. The experiment was terminated when the specimen deformed by 5.30 mm against 242 kN.

The R4 specimen suddenly exceeded the peak point of 725 kN and the center region was crushed at the same time on all four faces. Its reinforcement buckled under a deformation of 9.14 mm and an axial force of 438 kN, all of which were within the measurement zone. It was observed that a stirrup was broken under a displacement of 18.22 mm and a load of 187 kN. At the end of the experiment, 30 mm deformation was applied to the specimen. The R5 specimen exceeded the peak value under 694 kN and 3.29 mm deformation and suddenly crushed and collapsed. Crushing first started outside the measurement zone. However, when 538 kN and 5.52 mm displacement was read, the crushing was effective within the measurement zone. It was understood that 5 stirrups were broken during the axial force values of 187.2 kN, 110 kN, 82.9 kN, 73.9 kN and 73.3 kN respectively. The specimen was deformed by 30 mm during the experiment. There was no visible damage to the R6 specimen until the axial load value of 493 kN. As the load increased to 675 kN, crushing was observed outside the measurement zone. The first crushed area is the upper forehead of the specimen. This was followed by crushing of the upper and lower regions of one corner as the load increased to 797 kN. Under 4.05 mm deformation and 903 kN axial force, the specimen exceeded the peak point and collapsed due to sudden crushing inside the measurement zone.

The axial load strengths of the specimens are given in Table 4. The axial stiffnesses of the specimens were obtained from the slope of the output arms of the load-unit deformation curves of the experimental specimens. The ratios of axial stiffnesses diagonally between the specimens are given in Table 5.

**Table 4.** Experimental results

| Spec. | Axial load and Unit deformations | | | | | | | | Ratios | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Yield Point | | Peak Capacity | | | Ductility Limit | | | | | | |
| | $N_{yN}$* | $(\varepsilon_{cy})_i$ | $(\varepsilon_{cy})_o$ | $N_{oN}$* | $(\varepsilon_{co})_i$ | $(\varepsilon_{co})_o$ | $N_{0.85N}$* | $(\varepsilon_{0.85})_o$ | $\frac{N_{oN}}{N_{yN}}$ | $\frac{(\varepsilon_{co})_i}{(\varepsilon_{cy})_i}$ | $\frac{(\varepsilon_{0.85})_o}{(\varepsilon_{cy})_o}$ | $\frac{(\varepsilon_{0.85})_o}{(\varepsilon_{co})_o}$ |
| S1 | 647 | 0.0015 | 0.0047 | 807 | 0.0035 | 0.0083 | 686 | 0.0106 | 1.25 | 2.27 | 2.26 | 1.28 |
| S2 | 617 | 0.0016 | 0.0058 | 735 | 0.0028 | 0.0085 | 625 | 0.0110 | 1.19 | 1.74 | 1.89 | 1.30 |
| S3 | 776 | 0.0011 | 0.0048 | 961 | 0.0021 | 0.0070 | 817 | 0.0099 | 1.24 | 2.01 | 2.06 | 1.41 |
| S4 | 648 | 0.0013 | 0.0061 | 806 | 0.0021 | 0.0094 | 685 | 0.0114 | 1.24 | 1.53 | 1.88 | 1.22 |
| S5 | 583 | 0.0013 | 0.0061 | 725 | 0.0023 | 0.0093 | 616 | 0.0126 | 1.24 | 1.79 | 2.07 | 1.36 |
| S6 | 745 | 0.0014 | 0.0043 | 927 | 0.0024 | 0.0070 | 788 | 0.0096 | 1.25 | 1.71 | 2.23 | 1.36 |
| R4 | 600 | 0.0014 | 0.0047 | 756 | 0.0023 | 0.0078 | 643 | 0.0100 | 1.26 | 1.66 | 2.14 | 1.28 |
| R5 | 568 | 0.0015 | 0.0050 | 723 | 0.0030 | 0.0089 | 614 | 0.0106 | 1.27 | 2.00 | 2.11 | 1.19 |
| R6 | 780 | 0.0009 | 0.0048 | 944 | 0.0019 | 0.0068 | 802 | 0.0091 | 1.21 | 2.00 | 1.87 | 1.32 |
| M1 | 688 | 0.0010 | 0.0047 | 828 | 0.0023 | 0.0075 | 704 | 0.0091 | 1.20 | 2.39 | 1.92 | 1.22 |
| M4 | 663 | 0.0015 | 0.0046 | 825 | 0.0028 | 0.0074 | 701 | 0.0096 | 1.24 | 1.85 | 2.07 | 1.29 |

*Load values is in kN.4

**Table 5.** Comparative stiffness table of specimens with respect to each other

| | S1 | S2 | S3 | S4 | S5 | S6 | R4 | R5 | R6 | M1 | M4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| S1 | 1.00 | 1.01 | 2.01 | 1.21 | 1.19 | 1.26 | 1.17 | 0.96 | 1.62 | 1.74 | 1.26 |
| S2 | | 1.00 | 1.99 | 1.20 | 1.18 | 1.25 | 1.15 | 0.95 | 1.61 | 1.72 | 1.25 |
| S3 | | | 1.00 | 0.60 | 0.59 | 0.63 | 0.58 | 0.48 | 0.81 | 0.87 | 0.63 |
| S4 | | | | 1.00 | 0.98 | 1.04 | 0.96 | 0.79 | 1.34 | 1.43 | 1.04 |
| S5 | | | | | 1.00 | 1.06 | 0.98 | 0.81 | 1.36 | 1.46 | 1.06 |
| S6 | | | | | | 1.00 | 0.92 | 0.76 | 1.29 | 1.38 | 1.00 |
| R4 | | Undefined Region | | | | | 1.00 | 0.83 | 1.39 | 1.49 | 1.08 |
| R5 | | | | | | | | 1.00 | 1.69 | 1.81 | 1.31 |
| R6 | | | | | | | | | 1.00 | 1.07 | 0.78 |
| M | | | | | | | | | | 1.00 | 0.73 |
| M | | | | | | | | | | | 1.00 |

The unit deformation at the moment when the load-unit deformation curves of the experimental specimens were reduced to 85% of the maximum strength on the descending arm of the load-unit deformation curves was taken as the maximum unit deformation, and the specimen ductility was obtained from the ratio of this unit deformation to the unit deformation at yield. Accordingly, the ductility ratios obtained from the external measurement devices are presented in Table 6. In general, the specimens were not ductile enough. However, as it is known, excessive ductility is not expected in reinforced concrete columns subjected to concentric loading. Therefore, it is already known that this inadequacy in ductility is a behaviour of concentrically loaded reinforced concrete columns rather than the repair/strengthening method applied.

**Table 6**. Comparison of ductility ratios of specimens.

| | S1 | S2 | S3 | S4 | S5 | S6 | R4 | R5 | R6 | M1 | M4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| S1 | 1.00 | 0.99 | 0.87 | 1.06 | 0.89 | 0.97 | 0.88 | 0.97 | 1.04 | 0.90 | 0.97 |
| S2 | | 1.00 | 0.88 | 1.07 | 0.90 | 0.98 | 0.89 | 0.98 | 1.05 | 0.91 | 0.98 |
| S3 | | | 1.00 | 1.21 | 1.01 | 1.10 | 1.00 | 1.11 | 1.19 | 1.03 | 1.11 |
| S4 | | | | 1.00 | 0.84 | 0.91 | 0.83 | 0.92 | 0.99 | 0.85 | 0.92 |
| S5 | | | | | 1.00 | 1.09 | 0.99 | 1.09 | 1.18 | 1.01 | 1.09 |
| S6 | | | | | | 1.00 | 0.91 | 1.00 | 1.08 | 0.93 | 1.00 |
| R4 | | Undefined Region | | | | | 1.00 | 1.10 | 1.19 | 1.02 | 1.10 |
| R5 | | | | | | | | 1.00 | 1.08 | 0.93 | 1.00 |
| R6 | | | | | | | | | 1.00 | 0.86 | 0.93 |
| M | | | | | | | | | | 1.00 | 1.08 |
| M | | | | | | | | | | | 1.00 |

## 4. Conclusion

In this experimental study, the behavior of jacketed or reinforced rectangular columns (h/b=1.80 before jacketing and h/b=1.50 after jacketing) under monotonic axial loads was investigated. All members were jacketed in the unloaded condition. In addition, jacketing was applied after the deformations on the damaged member were reversed in repair and resurrection cases. As a result of the experiments, the following conclusions were reached by analyzing the parameters such as strength, stiffness, ductility and energy consumption obtained from the specimens.

• No difference was observed between the behavior of the new column obtained as a result of the repair/strengthening of rectangular section columns by jacketing and the repair/strengthening of square section columns by jacketing in previous studies and it was observed that both columns behaved the same.

• Although there was a 100% increase in cross-section and reinforcement, only the strength of the reinforced specimen reached 100% of the monolithic specimen strength. In the repaired members, 95% of the monolithic member strength was reached on average.

• The axial stiffness of all specimens was lower than the monolithic specimen stiffness. The reinforced specimen reached 90% of the monolithic specimen stiffness. In the repaired specimens, this ratio was around 60% to 70%.

• When the specimen ductility was compared with the monolithic column, the reinforced specimen ductility reached 80% of the monolithic column ductility. On the other hand, the repaired specimens could only reach 50~60% of the monolithic specimen ductility. In general, all specimens, including the monolithic, failed to show sufficient ductility with a behavior typical of reinforced concrete columns subjected to concentric loading.

• Although less than the monolithic specimen, the difference was very small and all specimens had sufficient energy dissipation capacity The findings and their implications should be discussed in the broadest context possible. Future research directions may also be highlighted.

• Even under laboratory conditions, where meticulous work is required, ensuring the sufficient success of the jacketing application necessitates rigorous supervision and should be entrusted to experienced personnel to achieve the desired quality and performance.

• The strength reduction observed at the tail end of the experimental load-strain curves for monolithic, strengthened, repaired, and retrofitted elements, at approximately 0.03 strain, is around 60%.

**Author Contributions**

The percentages of the author(s) contributions are presented below. All authors reviewed and approved the final version of the manuscript.

| | Y.K. | H.C. |
|---|---|---|
| C | 0 | 100 |
| D | 10 | 90 |
| S | 0 | 100 |
| DCP | 100 | 0 |
| DAI | 40 | 60 |
| L | 60 | 40 |
| W | 90 | 10 |
| CR | 50 | 50 |
| SR | 100 | 0 |
| PM | 50 | 50 |
| FA | 50 | 50 |

C=Concept, D= design, S= supervision, DCP= data collection and/or processing, DAI= data analysis and/or interpretation, L= literature search, W= writing, CR= critical review, SR= submission and revision, PM= project management, FA= funding acquisition.

**Conflict of Interest**

The authors declared that there is no conflict of interest.

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