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Axial Compressive Behavior of Square Concrete Columns Externally Collared by Light Structural Steel Angle Sections

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Abstract
The buildings which were designed and built in the past according to the old seismic code or no seismic code at all urgently require a retrofitting effort to satisfy the current seismic and building codes. To demolish and rebuild them would be very costly than to retrofit. Some of the available seismic retrofitting techniques are the concrete jacketing, steel jacketing, external strand pre-stressing, Fiber Reinforced Polymer (FRP) jacketing, and steel collar jacketing. In this study, a new steel collar is proposed as an alternative of seismic retrofitting techniques of existing buildings. The primary objective of the study is to propose an economical, efficient, effective, and practical method for retrofitting the square or even rectangular RC columns. Light structural steel angle sections have been introduced for collar elements. These collars will be installed externally at the perimeter of RC column sections to enhance their strengths and ductilities. To achieve this objective, a set of experimental work was carried out to investigate the effectiveness of the proposed technique in retrofitting the existing RC column specimens. Eight column specimens were casted and tested under monotonic compressive load with or without the collars in order to investigate the performance of the proposed retrofitting method. The volumetric ratio of steel collar elements is set as a main parameter in the study. Three control specimens were only confined with conventional stirrups for comparison. The remaining specimens were just confined with the external steel collars. The test results indicate that the proposed external retrofitting technique works well for confinement method. Significant strength and ductility gains are observed in the study. The proposed external steel collars as an alternative of retrofitting techniques for existing columns can be concluded to be very promising.

Keywords: compressive strength, ductility, external confinement, retrofit, square concrete columns, steel collars, stress-strain curves.

Introduction

In the past, the urgency of designing seismic-resistant buildings was not mandatory. If a seismic code provided the seismic map was available at the time, the designed seismic load produced according to the code was lower than that obtained by the current seismic code. This is due to the lack of knowledge on seismic risk in the past that has not been well assessed and studied. Most of the existing buildings were designed according to the old seismic code or even no seismic code at all in the old time. They are currently required urgently to be enhanced in terms of strength and performance. The later has gained so much attention to be the basis of the nowadays seismic design concept which does not require the strength anymore but the ductility. Thus, buildings which were designed and built in the past in accordance with the old seismic code or no seismic code urgently require a retrofitting effort to satisfy the current seismic and building codes. The main reason is that they do not have enough strength or ductility as per the latest seismic and building codes. To demolish and rebuild them would be very costly than to retrofit. The word of retrofitting has been increasingly popular than strengthening since highly likely to meet the current seismic and building code provisions the ductility aspect would govern over the strength. The strength in a ductile seismic building structure is no longer a primary concern anymore just like in the old time or when it is under gravity load.

Ductility in structures is arguably very important parameter, especially on structures in high seismic regions. The lack of ductility will lead to undesired failure mechanism of the structures. In concrete members, providing confinement has been known to improve the overall behavior. The improved behavior generally includes the enhanced strength and ductility [1-7]. Many researches have been conducted to study the effect of confinement, which can be found in many literatures [1-36]. With development of the knowledge about seismic action, typically newer codes specify higher demand. One implication of this fact is the needs of upgrading many existing structures, including Reinforced Concrete (RC) structures. According to Liu et al. [8], the resulting deficiencies that often characterize old existing RC frame structures include: (1) insufficient transverse reinforcement to confine the column core and to restrain buckling of longitudinal reinforcement; (2) inadequate lap splices located immediately above floor levels where inelastic actions may be concentrated with large flexural demand; (3) insufficient shear strength to develop the column flexural capacity, or the potential degradation of column shear strength with increasing flexural ductility demand; (4) inadequate column strength to develop a strong-column weak-beam mechanism, and (5) deficient beam-to-column joint dimensions and details. For most framed structures, it is more economical to design for dissipating seismic energy in a flexural mode by forming plastic ductile hinges in beams rather than in columns [9]. However, columns are critical elements in any structural building system and their performance during a seismic event can dominate the overall performance of the structure since single column failure can lead to additional failures and potentially result in total building collapse [8]. Mander et al. [2, 3] mentioned that the most important issue in plastic hinge design of reinforced concrete columns is the availability of sufficient transverse reinforcement for confining the concrete, preventing the buckling of longitudinal reinforcement, and avoiding brittle shear failure. In order to provide such requirement on existing deficient RC columns, retrofitting should be introduced.

One of the commonly used retrofitting approaches of RC columns is by improving the confinement. The most conventional method is by installing the additional reinforcement embedded in the concrete jacketing. The more recent methods are by externally introducing the use of confinement layer or elements since they are considered easier and faster to implement as a retrofitting techniques to the existing buildings. Among this approaches are the steel sheet jacketing [10-13], fiber-reinforced polymer (FRP) composite jacketing [14-17], and steel collar jacketing [8, 18-23]. Ideally, an effective retrofitting technique shall possess such characteristics as being easy to implement, minimizing disruption to the use of the structure, not requiring highly specialized skills, minimizing labor costs, and resulting in efficient performance [8]. Concrete and steel jacket are very effective but inconvenient to install, because doing so requires using scaffolds for curing the concrete or grout [13]. FRP jackets have several advantages over the steel and concrete jackets: (1) ease of installation; (2) no increment of the cross section; and (3) no increment of the flexural or shear stiffness of the structure. However the price of FRP jackets is generally higher than that of concrete and steel jackets [13]. Driver et al. [8, 18-19] developed retrofitting technique by using steel collars cut from solid steel plates or Hollow Square Sections (HSS) which are installed by using high strength bolts or welding. The method has been proven to be effective. Pudjisuryadi et al. [20, 21], Pudjisuryadi and Tavio [22], and Tavio et al. [23] reported their early analytical as well as experimental works on the investigations of the effects of external steel collars on square concrete columns. The confinement elements used were a set of light structural steel angle section collars, installed by fastening the corner bolts without the application of any grouts. This paper further discusses the performance of the retrofitting method in order to provide a better understanding.

Experimental Setup

A set of column specimens collared by light structural angle steel sections was tested to study their behaviors under axial compression and to confirm that the proposed technique is a very potential and promising option for retrofitting square or possibly rectangular RC columns, particularly when the confinement requirement is not satisfied or provided. The cross sectional dimensions and height of the specimens were $200 \times 200 \times 600$ mm, respectively. The specimens were divided into 400-mm middle test region and two 100-mm strengthened regions at both ends. The clear concrete cover was 20-mm thick. The strengthened regions were better confined than the test region such that no failure was possible to occur in these regions. The square concrete columns were then confined externally by a set of light structural steel angle section collars. The angle sectional dimensions were $40 \times 40 \times 4$ mm. The external confinement was attached on the column specimen by installing the steel collars on its four faces with uniform spacing and then fastened the structural bolts at its four corners. The perspective illustration of the assembled and exploded views of a typical column specimen (Specimen S03 is used for illustration) can be seen in Figures 1 and 2. The 400-mm mid-test region was confined with various numbers of external steel collars. A set of rods were also installed within the test regions defining the gage lengths on each face of the column specimens. Layout plan and elevation views of the typical specimens are shown in Figures 3 and 4. The column specimens were then tested under monotonic concentric compressive loading as shown in Figure 5. Universal testing machine was used to perform the compression test. Four load cells (with capacity of 50 tons each) were placed below the test specimens (Figure 6). Three 60-mm thick plates were also placed on the load cells to ensure uniform load distributions. The column specimens were then placed on the steel plates at which the axial axis of the specimen coincided with the center of four load cells. Four Linear Variable Differential Transducers (LVDTs) were also installed at each column specimen face to measure the axial deformation during the compressive test (see Figure 7). All load cells, LVDTs, and strain gages were connected to a Data Logger system and further extended to the notebook computer for measuring all the data needed such as loads, deformations, and strains during the test.

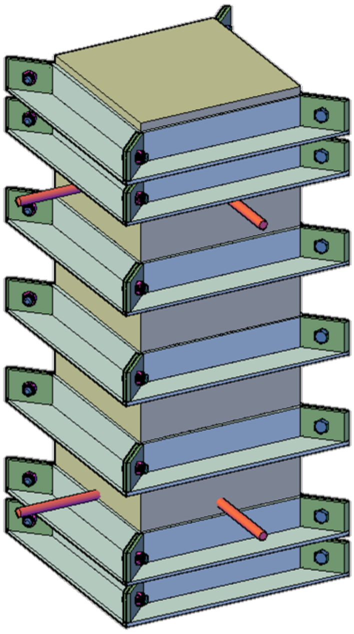


Figure 1: Typical Perspective View of Test Specimen (Specimen S03 as an Illustration)

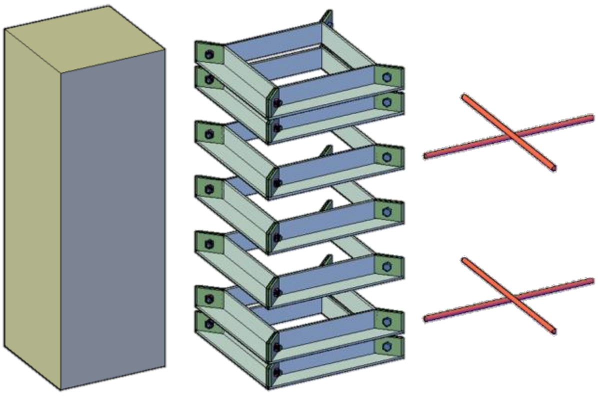


Figure 2: Typical Exploded View of Test Specimen (Specimen S03 as an Illustration)

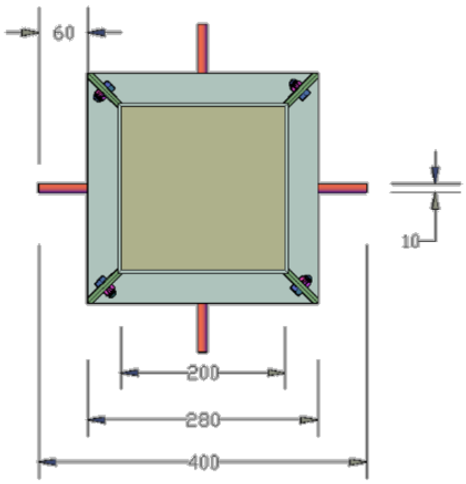


Figure 3: Typical Layout Plan View of Test Specimen

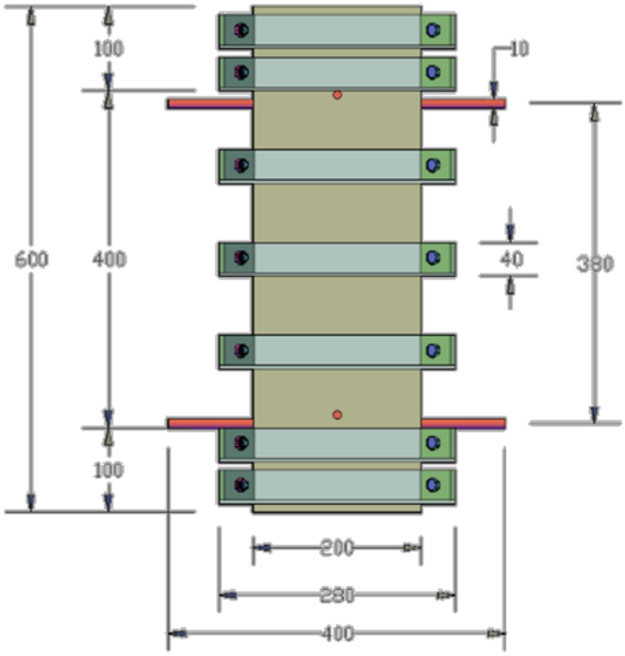


Figure 4: Typical Elevation View of Test Specimen (Specimen S03 as an Illustration)

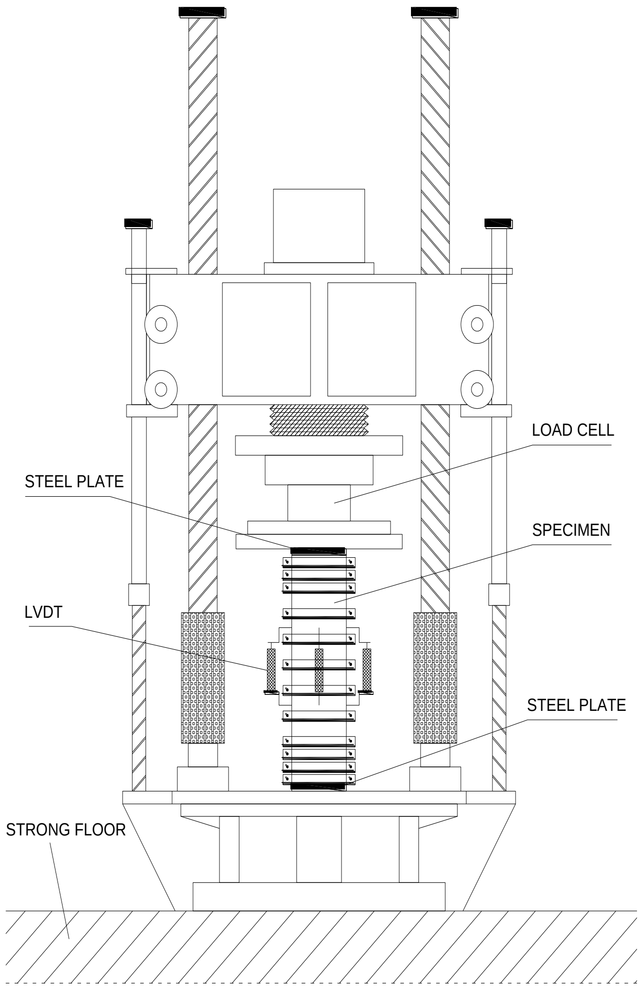


Figure 5: Monotonic Compressive Loading Test Setup

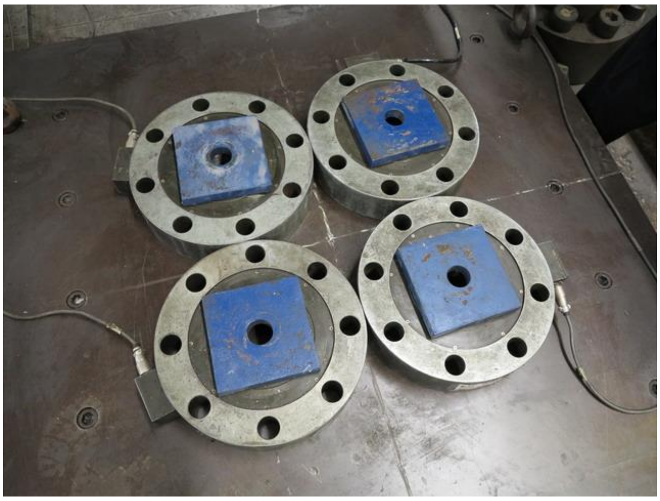


Figure 6: Four Load Cells with 50 Tons Capacity Placed Below Test Specimen

Figure 7: Typical Test Setup (S04 is Shown)

Specimen Design

Three control specimens were made, namely CS01, CS02a, and CS03a. These control specimens are intended to study the behavior of conventionally confined column specimens under compressive load. CS01 is constructed without any confinement within the test region, and only 4-D10 longitudinal reinforcements are used. CS02a is designed to represent the condition of columns confined without seismic provisions. The confinement requirements according to Indonesian concrete code are as follows:

$$A_{vmin} = \frac{75\sqrt{f_c'}}{1200}\frac{b_w \cdot s}{f_{yh}} \geq \frac{1}{3}\frac{b_w \cdot s}{f_y} \quad (1)$$

where:

A_{vmin} = minimum area of stirrups,
b_w = width of concrete element,
f_c' = concrete strength,
s = spacing of stirrups,
f_{yh} = yield strength of stirrups steel.

This requirement should be accompanied with maximum shear spacing, which is the smallest of these expressions:

a) 16 times the diameter of longitudinal reinforcement (160mm)
b) 48 times the diameter of transverse reinforcement (480mm)
c) the smaller dimensions of the column (200mm)

Due to test region length restriction, a confinement of D10-133 mm is selected (A_v/s = 1.18 mm^2/mm, and volumetric ratio = 0.89 percent). The volumetric ratio is defined as the volume of confining steel with respect to the volume of concrete. It should be noted that normally the volume of concrete for calculating volumetric ratio is the confined concrete core. But, since the confined concrete sectional area is different in the case of internal (inside core) and external confinement (gross area), either should be picked for the sake of comparison. In this paper, the volume of concrete for calculating the volumetric ratio is determined from the gross cross-sectional area of the column multiplied by the spacing of the confining steel in the mid-test region. Specimen CS03a is designed to represent the condition of columns confined with seismic confinement requirement. The seismic confinement requirements according to Indonesian concrete code are as follows:

$$Av_{min1} = 0.09\left(s \cdot h_c \cdot \frac{f_c'}{f_{yh}}\right) \quad (2)$$

$$Av_{min2} = 0.3\left(s \cdot h_c \cdot \frac{f_c'}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}}-1\right) \quad (3)$$

where:

h_c = cross sectional dimension of member core
A_g = gross area of concrete section
A_{ch} = cross sectional area of member core

This requirement must be accompanied with maximum shear spacing specified by Indonesian concrete code, which is the smallest value of these following expressions:

a) one quarter of smallest dimensions of column
b) six times the diameter of longitudinal bars
c) 100+(350 –h_x)/3 < 150mm
d) 100mm

where h_x is the maximum center-to-center distance of supported longitudinal bars. Reinforcing confinement of D10-50 is chosen for this specimens (A_v/s = 3.14 mm2/mm, and volumetric ratio = 2.36 percent). Illustration of Specimens CS01, CS02a, and CS03a can be seen in Figure 8. Since the axial loading is concentric, all sides of column specimens should suffer the same strains (stresses). However, in order to capture the unexpected eccentricity or other imperfection of the specimens, several strain gauges are attached to longitudinal and stirrups inside the specimens.

Another five specimens are built exactly like specimen CS01. These five specimens are then externally retrofitted by sets of steel angle collars. The amount of steel collars are varied from only one steel collar within the middle test region (S01), and then increased by one steel collar each time until totally five steel collars mounted to the last specimen (S05). The five specimens are named S01, S02, S03, S04, and S05. Without any internal confinement steel in the test region, these five specimens were built to study the basic effect of proposed confinement technique on square concrete columns. The resulting volumetric ratios of these five specimens range from 3.84 to 11.51 percent. Illustration of these five specimens can be seen in Figure 9 to Figure 13. Besides the longitudinal bars inside, several strain gages are also attached on the steel collars. Table 1 summarizes the details of reinforcement and confinement of the test specimens.

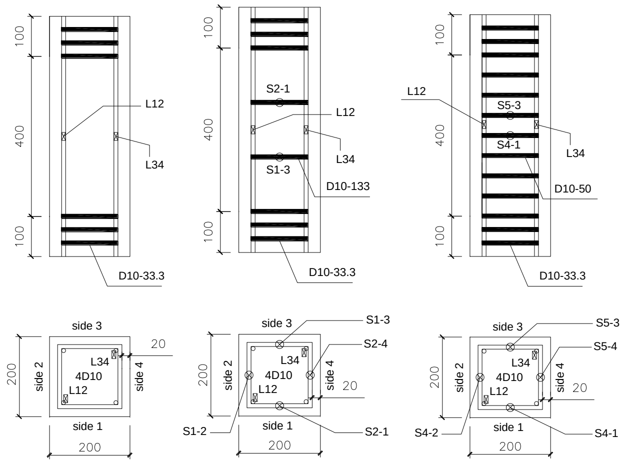


Figure 8: Control Specimens CS01, CS02a, and CS03a

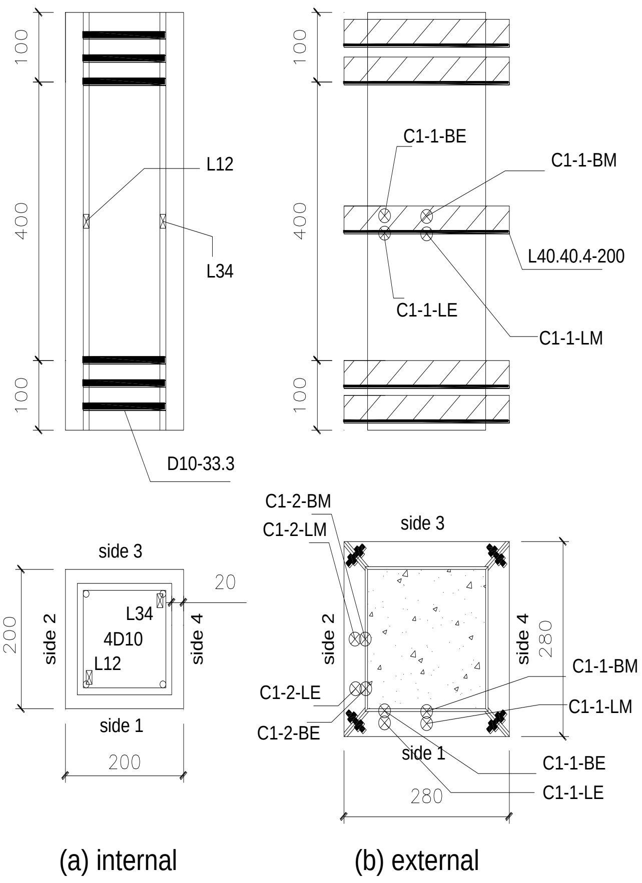


Figure 9: Test Specimens S01

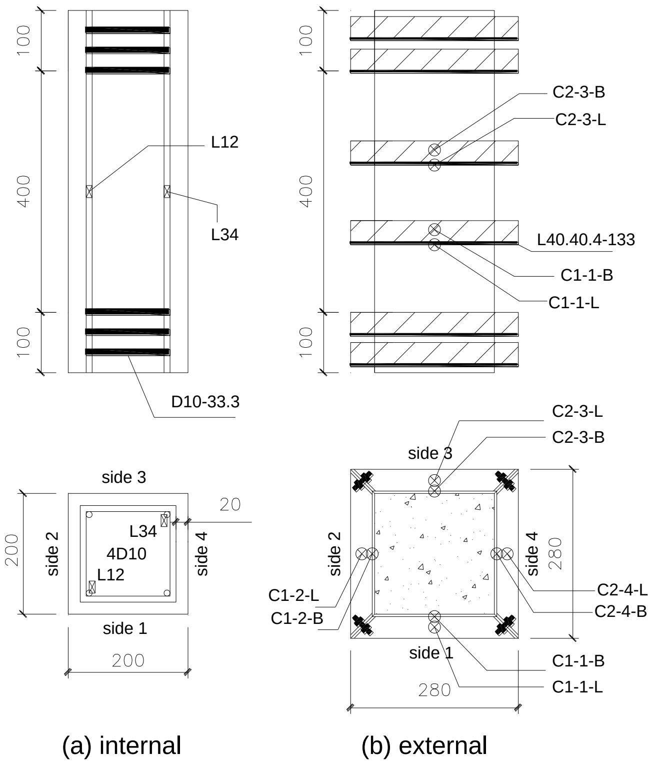


Figure 10: Test Specimens S02

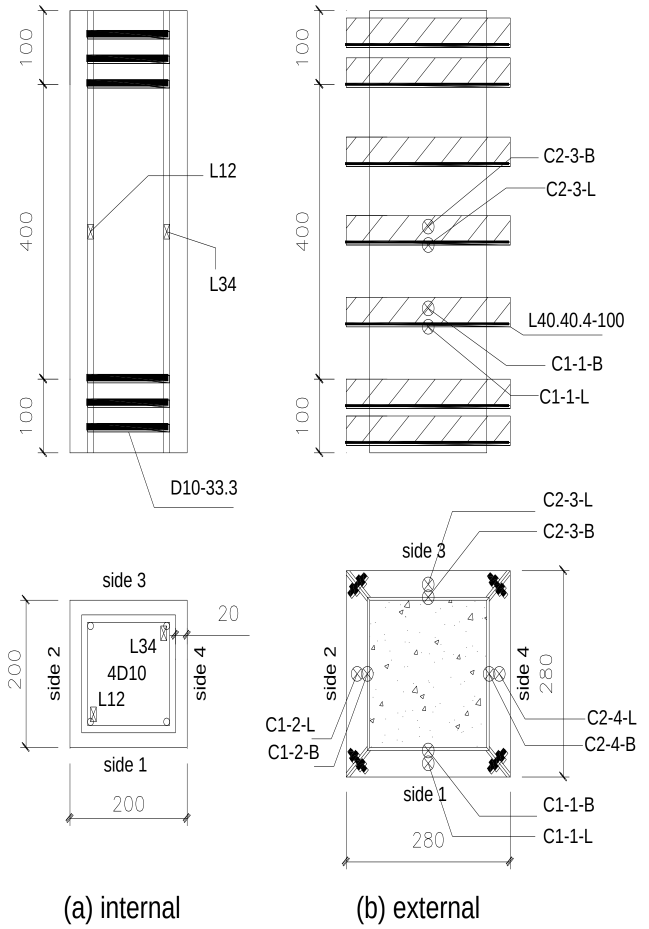


Figure 11: Test Specimens S03

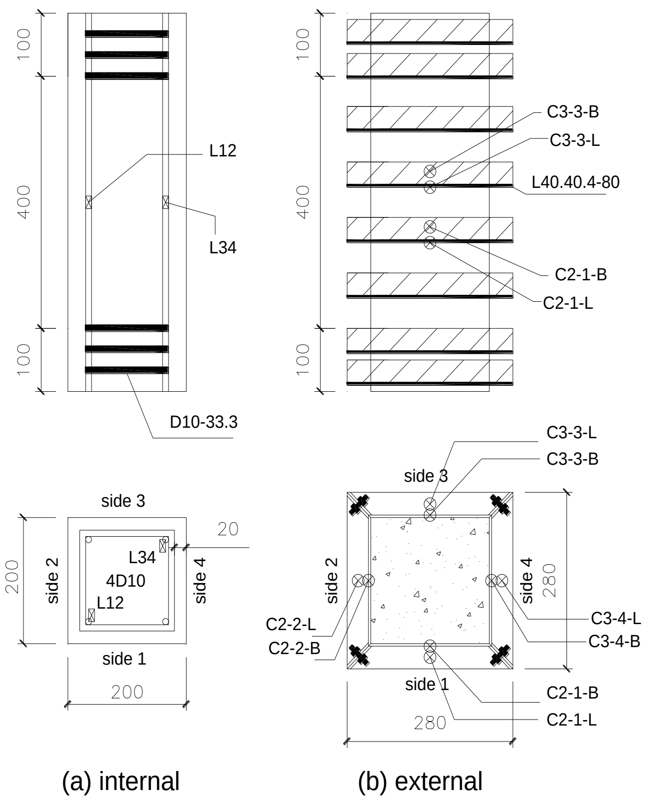


Figure 12: Test Specimens S04

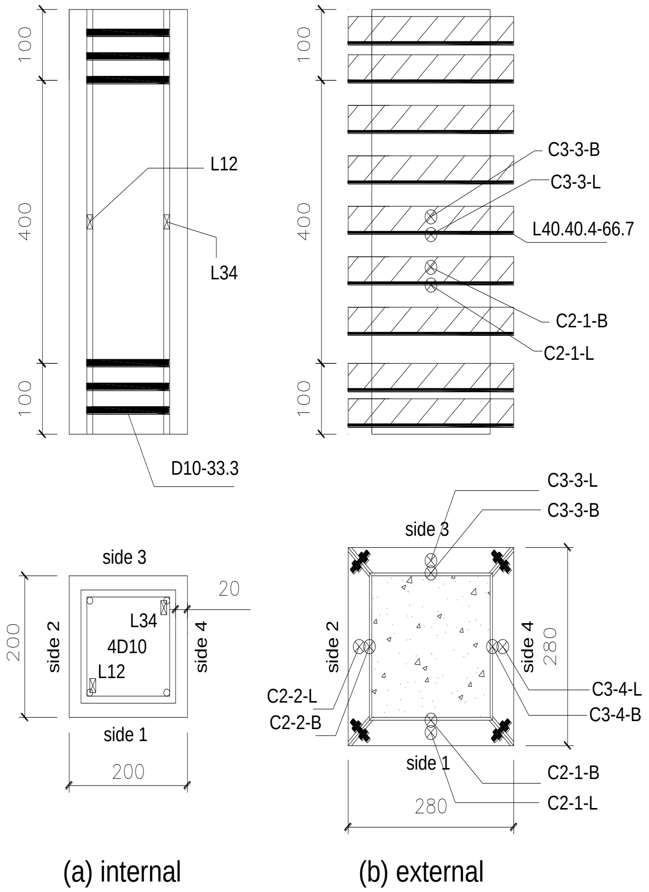


Figure 13: Test Specimen S05

Table 1: Details of Test Specimen's Reinforcement and Confinement

Label	Long Bars	Confinement Elements
CS01	4 D10	None
CS02a	4 D10	D10-133 (vol. ratio = 0.89%)
CS03a	4 D10	D10-50 (vol. ratio = 2.36%)
S01	4 D10	L40.40.4-200 (vol. ratio = 3.84%)
S02	4 D10	L40.40.4-133 (vol. ratio = 5.77%)
S03	4 D10	L40.40.4-100 (vol. ratio = 7.68%)
S04	4 D10	L40.40.4-80 (vol. ratio = 9.60%)
S05	4 D10	L40.40.4-67 (vol. ratio = 11.34%)

Results and Discussions of Monotonic Compressive Load Test

The mechanical properties of the concrete used in the test specimens were obtained from standard cylinders made from the same mix proportion. The average strength (f_c') of totally 11 cylinders is 23.93 MPa with a standard deviation of 2.01 MPa. Tensile tests were carried out to obtain the mechanical properties of steel bars and steel angle sections. The average yield strength (f_y) of deformed bars (with nominal diameter of 9.5 mm) used in the test specimens is 317 MPa with a standard deviation of 5.9 MPa obtained from three bar samples. Tensile test of a strip steel plate, cut from the steel angle section, indicated that the average yield strength (f_{ysc}) is 285 MPa. Monotonic compressive load tests of all specimens were conducted with controlled axial displacement, and the axial resistances of the columns were recorded. The tests were stopped if one of the following criteria was found: (1) failure of specimen; (2) resistance drops below 50 percent of the peak strength; or (3) limitation of LVDT capacity.

Strength and Ductility Improvement

Two of the main interests by providing confinement on concrete columns are the strength and ductility improvement. In order observe those, a normalized axial stress-strain curves are presented (see Figure 14). From control Specimen CS01, it is found that the concrete strength is equal to 17.02 MPa ($f_{c0}^{'}$). This strength is used to normalize the stress-strain curve in order to investigate the effect of confinement of other specimens. The peak strain (ε_{01}) and ultimate strain ($\varepsilon_{cu}' = \varepsilon_{f50}$) of Specimen CS01 are equal to 0.23 percent and 1.37 percent, respectively. In this paper, the strain corresponding with 50 percent of peak strength on the descending branch of the curve is defined as the ultimate strain. To identify the ductility of axially loaded specimens, the commonly used parameter is the relative strain ductility ratio ($\mu_\varepsilon = \varepsilon_{f85}/\varepsilon_{01}$). ε_{f85} is defined as the strain corresponding with 85 percent of peak strength on the descending branch. The numerical data result of all specimens is listed in Table 2.

The strength gain can directly be indicated by the normalized peak strengths of Specimens. CS02a which was conventionally confined with deficient volumetric ratio, showed no strength gain (0.95). The control Specimen CS03a showed strength gain of 1.21 due to its good confinement. The collared Specimens S02 and S04 seemed to have a little deviated strength gain. Specimens S02 showed a bit high strength gain (1.32), while S04 show a bit low (1.23). The other collared Specimens S01, S03, and S05 showed a good strength gain pattern (1.08, 1.21, and 1.42 respectively). In term of ductility, it can be seen that CS01, CS02a showed very brittle behavior, that the strength decreased rapidly after reaching the peak strength (μ_ε = 1.63 and 3.27 respectively). S01 showed rather similar behavior, except that it had late post-peak ductility response (μ_ε = 2.30). CS03a showed good ductility (μ_ε = 15.55) until it finally lose the strength at about 10.90 percent axial strain. Collared specimens with higher volumetric ratio, better ductility pattern is observed except for specimen S04 which suffered early steel collar failure. Specimens S02, S03, and S05 indicated μ_ε of 4.84, 8.15, and 26.16 respectively, while S04 only showed μ_ε of 3.46. In term of ductility, the proposed retrofitting method has demonstrated that it can provide comparable value as the conventionally confined Specimen CS03a which was built according to seismic provisions.

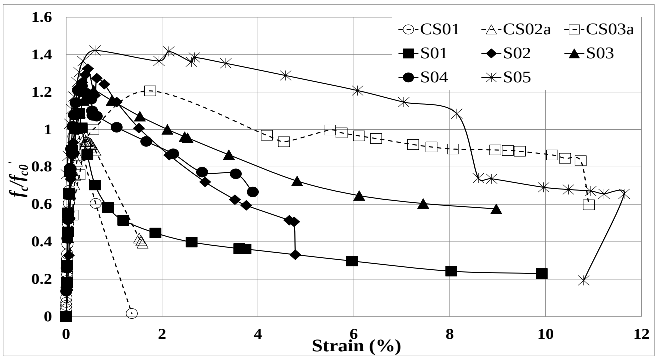


Figure 14: Normalized Axial Stress vs. Strain of Test Specimens

Table 2: Summary of Monotonic Compression Tests

Parameter	CS01	CS02a	Cs03a	S01	S02	S03	S04	S05
P_{cmax}-kN	676	645	815	733	896	817	833	961
ε_{cc} (%)	0.23	0.38	1.75	0.26	0.45	0.57	0.33	1.83
ε_{f85} (%)	0.38	0.76	3.61	0.53	1.12	1.89	0.80	6.07
ε_{f50} (%)	1.37	1.57	10.9	1.86	3.76	8.97	3.89	10.8
$\mu_\varepsilon = \varepsilon_{f85}/\varepsilon_{01}$	1.63	3.27	15.6	2.30	4.84	8.15	3.46	26.2
f_{cc}' (MPa)	17.0	16.2	20.5	18.5	22.6	20.6	21.0	24.2
f_{cc}'/f_{c0}'	1.00	0.95	1.21	1.08	1.32	1.21	1.23	1.42

Notes:

P_{cmax} = maximum resistance contributed by concrete
ε_{cc} = axial strain corresponding to P_{cmax}
ε_{01} = ε_{cc} of unconfined specimen (CS01)
ε_{f85} = strain corresponding to 85 percent of P_{cmax} on the descending curve
ε_{f50} = strain corresponding to 50 percent of P_{cmax} on the descending curve
f_{cc}' = confined concrete strength
f_{c0}' = concrete strength of unconfined specimen (CS01)

Failure Mechanism of the Specimens

From the strain measurement, it is evident that the stirrups, as well as the steel collars acted as confinement element. While the longitudinal bars were in compression, the stirrups and steel collars were in tension during the test. Typical strain from the longitudinal bars (shown for Specimen CS01) can be seen in Figure 15. The typical strain in the stirrups (shown for Specimen CS03a) can be seen in Figure 16. Typical strain in steel collar (shown for collar 3 of Specimen S05) can be seen in Figure 17. The images of the damaged Specimens CS01, CS02a, and CS03a after the tests can be seen in Figure 18. It is obvious that the absence of any confinement (CS01) caused brittle diagonal failure in the specimen. Arbitrary crack initiation would progress rapidly which lead to sudden failure of this unconfined specimen. Specimen CS02a which was poorly confined also suffered brittle failure, but the damage was not as severe as CS01. Specimen CS03a which is confined conventionally by stirrups required by seismic provisions could prevent the core from severe brittle failure. Buckling of longitudinal bars was observed, but it should be noted that it happened at a very large axial strain (the test was stopped at axial strain more than 10 percent).

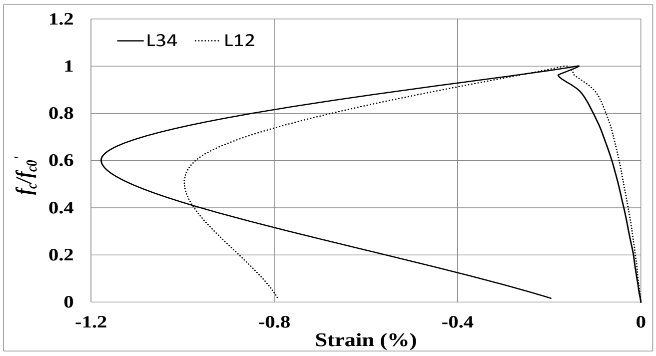


Figure 15: Normalized Axial Stress vs. Strain of Longitudinal Bars (CS01)

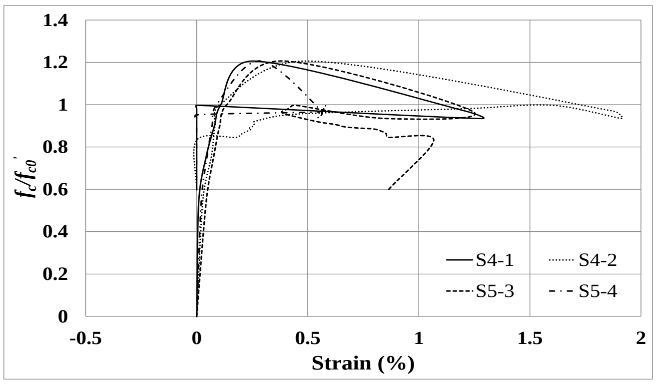


Figure 16: Normalized Axial Stress vs. Strain of Stirrups (CS03a)

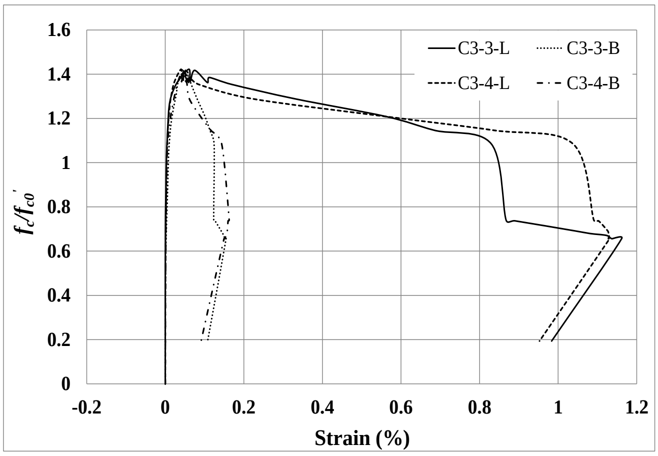


Figure 17: Normalized Axial Stress vs. Strain of Collar 3 (S05)

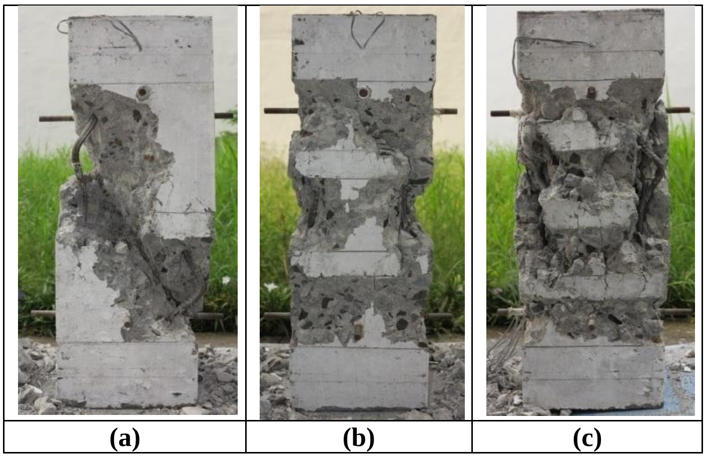


Figure 19: Damaged Specimens: (a) S01, (b) S02, and (c) S03

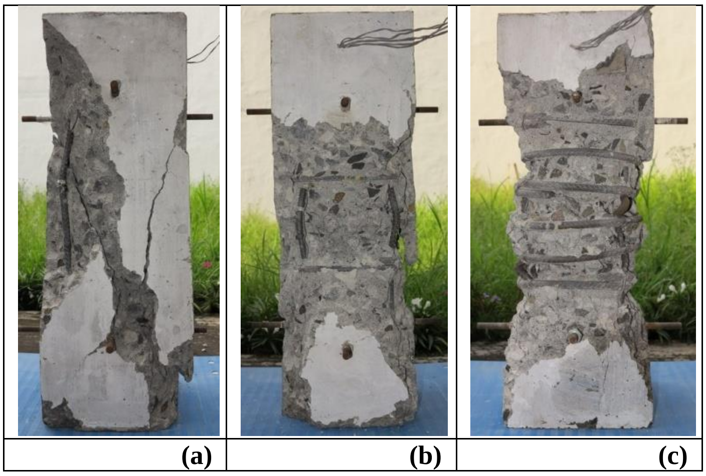


Figure 18: Damaged Specimens: (a) CS01, (b) CS02a, and (c) CS03a

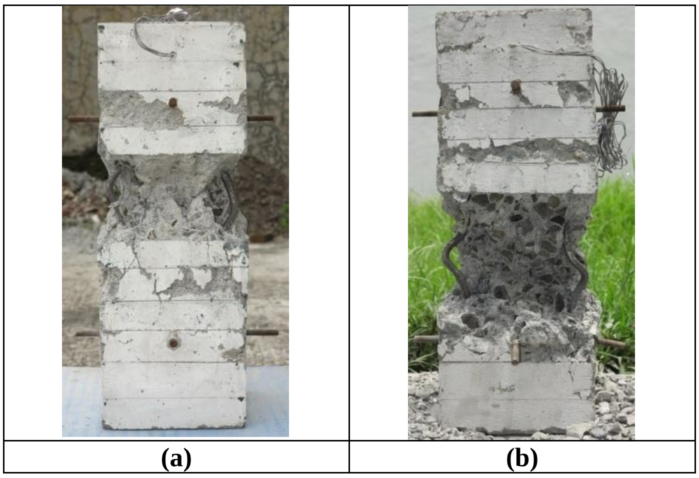


Figure 20: Damaged Specimens: (a) S04, and (b) S05

The concrete damages of collared specimens after the tests can be seen in Figure 19 and Figure 20. The lightest confined Specimen S01 showed brittle diagonal failure (see Figure 19(a)). It is evident that the steel collar did not fully utilize its function as confinement element that only small deformation was observed (see Figure 21). Damage patterns in Specimen S02 clearly showed the confinement effect of the steel collars that severe concrete damages occurred in regions in between the collars (see Figure 19(b)). But still, the concrete failure occurred prior to full confining potential of the collars (again, the collars showed slight deformations as seen in Figure 22). In Specimen S03, the brittle failure is completely avoided. The specimen could still maintain half of its peak axial capacity at a very large axial strain of 8.97 percent. It can be seen in Figure 19(c) that the specimen was severely damaged (spall of concrete and buckling of longitudinal bars). But the fact that it still has good resistance showed that the confinement worked and protected the inner core. Out of the three collars installed (see Figure 23 and Figure 24), the one at the middle of the test region (collar 2) worked most optimally that it experienced the most deformation.

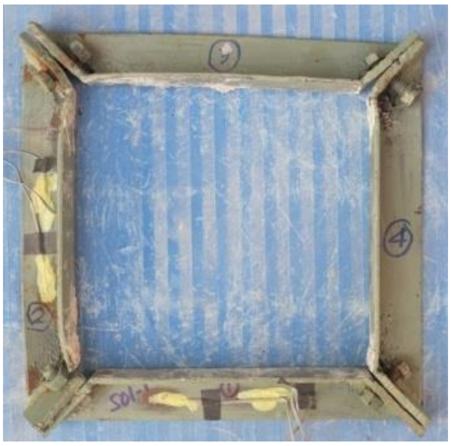
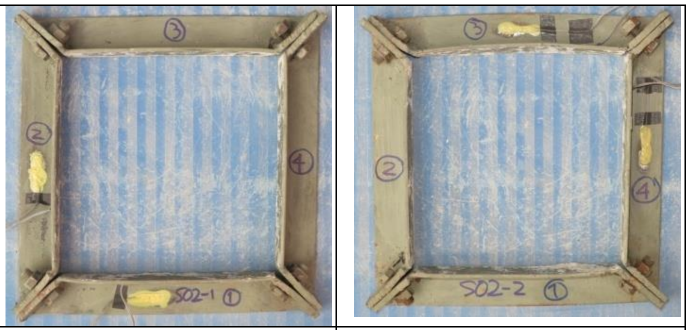


Figure 21: Damaged Collar 1 (S01)

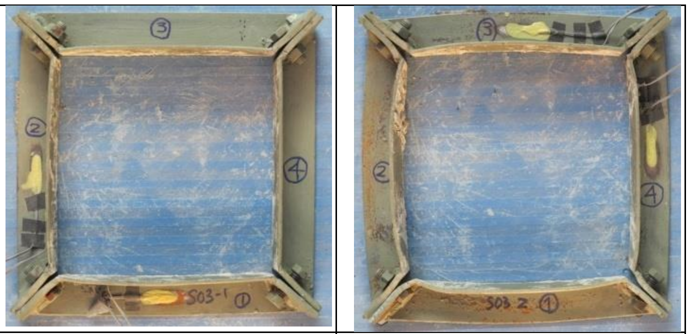
Unfortunately, S04 failed to show the expected performance. Due to imperfection of workmanship in preparing the collar, welding at one of the corner suffered early failure (Figure 25). Severe concrete damage was observed at the level of failed steel collar (Figure 20a).

Specimen S05 showed very good performance as expected. The post peak behavior show minor degradation up to large axial strain of about 8 percent. At this point, one collar failed and some concrete damage occurred. But since it still can maintain about 50 percent of its peak capacity, the test was continued. The specimen finally lost its resistance when the second collars failed at axial strain almost as large as 12 percent. The failed steel collars are shown in Figure 26. Important notes on the observation of the specimen damages are summarized in Table 3.



(a)	(b)

Figure 22: Damaged Collars: (a)1; and (b)2 of S02

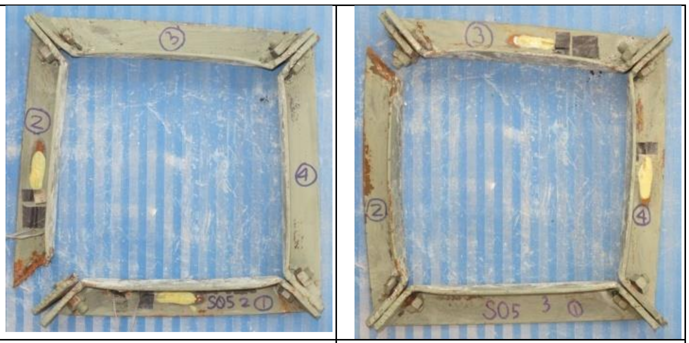


(a)	(b)

Figure 23: Damaged Collars: (a)1; and (b)2 of S03

Figure 24: Damaged Collar 3 (S03)

Figure 25: Welding Failure of Collar 3 (S04)



(a)	(b)

Figure 26: Damaged of Two Failed Collars: (a)2; and (b) 3 of S05

Table 3: Important Notes on Experimental Tests of Column Specimens

Specimen	f_{cc}'/f_{c0}'	Remark (descending branch)
CS01	1	Strength loss after descending to 60% of peak strength (at strain 0.62%). Brittle diagonal failure and buckling of longitudinal bars were observed.
CS02a	0.954	Test was stopped after descending branch dropped below 50% of peak strength at strain about 1.5%. Excessive damages and buckling of longitudinal bars were observed.
CS03a	1.206	Test was stopped at 50% peak strength (strain 10.90%), coinciding with LVDT limitation. Still can resist axial force, but buckling of longitudinal bars was observed.
S01	1.085	Strength dropped below 50% at strain about 1.2 %. Brittle diagonal failure and buckling of longitudinal bars were observed.
S02	1.325	Strength dropped below 50% peak strength at strain about 3.5%. Buckling of longitudinal bars was observed.
S03	1.209	Strength dropped below 50% peak strength at strain about 7.4%. Buckling of longitudinal bars was observed.
S04	1.232	Strength dropped below 50% peak strength at strain about 3.8%. Failure of collar 3 and buckling of longitudinal bars were observed.
S05	1.422	Two strength drops at 76% of peak strength (strain 8.15%), and at 46% of peak strength (strain 11.64%) due to broken collars 2 and 3 respectively. Buckling of longitudinal bars was also observed.

Concluding Remarks

As alternative of the already available retrofitting method, an external confining technique of square concrete columns is proposed. The method uses a set of steel angle collars as external confinement on square concrete columns. Some advantages of the proposed method are better constructability by introducing economic light structural steel angle sections which only involves minor cutting and welding processes, and has higher applicability by only mounting up the collars on the four faces of the column without any grouting and then fasten the structural bolts at its four corners. An experimental work has been conducted to validate the reliability of the proposed technique. From the test results, some conclusions can be made as follows:

- Improved axial stress-strain behavior was achieved by specimens externally confined by the proposed method as compared to the plain concrete control specimen (CS01).
- Specimens with smaller amount of steel collars suffered brittle failure, but ductile behaviors were observed in specimens with larger amount of steel collars. It also should be noted that specimens with small amount of confinement were more likely to experience un-symmetric failure (diagonal crack seen in S01). In the case of S01, symmetric resistance (measured from lvdt reading on each side of specimen) was only observed up to 80 percent of ascending branch. From that point, damage started leading to unsymmetric resistance of the specimen.
- From damaged patterns observation, it is clear that the steel collars work as confining element. Strips of concrete regions confined by the steel collars show less damaged regions in between the steel collars.
- Behavior of control specimen CS03a with 2.36 percent volumetric ratio of internal confining element, is matched by steel collared specimens S03 (with 7.68 percent volumetric ratio of confining element). Both specimens can reach peak strength about 1.2 times of CS01 strength, and show axial strain at 50 percent of peak strength on descending curve (ε_{f50}) more than 8.00 percent.
- The most heavily steel collared specimen S05 (with 11.34 percent volumetric ratio of confining element) can reach peak strength of 1.422 times of CS01 strength, and show ε_{f50} more than 10.00 percent.
- The failures of steel collars were often located in the corners. Improvement of corner plates and welding works were encouraged for better performance of the steel angle collars as external confinement element.

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