

Performance of Externally Steel Angle-Collared Concrete Columns Under Axial Compressive Load

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Abstract – Deficient Reinforced Concrete (RC) columns and higher demands of new codes are the main reasons for retrofit works for RC columns. One of the deficiencies involves the lack of confining elements which leads to poor ductility. Confining stress conventionally provided by transverse reinforcement has been recognized to enhance the strength and ductility of RC columns. Nowadays, the confinement method has been further developed to be applied externally which is more suitable for retrofit works. In square or rectangular columns, providing effective confining stress is not as simple as that of circular columns. In this research, displacement controlled compressive test on a set of square RC columns retrofitted by steel angle collars is presented. The volumetric ratio of the confining elements is the main parameter to investigate the effectiveness of the retrofitting technique. Enhancement of axial strength and ductility are observed as the confining element volumetric ratio increases. **Copyright © 2024 Praise Worthy Prize S.r.l. - All rights reserved.**

Keywords: Analytical Model, Disaster Risk Reduction, External Retrofit, Square RC Columns

Nomenclature

ε_c	Strain of concrete
ε_{cc}	Axial strain corresponding to P_{max}
ε'_{cc}	Confined concrete compressive strain
ε'_{c0}	Unconfined concrete compressive strain
ε_{Pmax}	Axial strain corresponding to P_{max}
E_c	Plain concrete modulus of elasticity (MPa)
E_{sec}	Confined concrete modulus of elasticity (MPa)
f'_{cc}	Confined concrete compressive strength (MPa)
FRP	Fiber-Reinforced Polymer
P_{max}	Total column's maximum axial capacity
P_{cmax}	Concrete column's maximum axial capacity
r	Constant
RC	Reinforced Concrete

I. Introduction

As seismic activity has been better understood, codes have become more demanding, necessitating the reinforcing and retrofitting of many older Reinforced Concrete (RC) structures. Liu et al. [1] state that the following deficiencies typically result from old, existing RC frame structures: (1) insufficient shear strength to develop the column flexural capacity or the possibility of increasing flexural ductility demand causing the column shear strength to degrade; (2) insufficient column strength to develop a strong-column weak-beam mechanism; and (3) insufficient beam-to-column joint dimensions and details. Designing for the flexural mode of seismic energy dissipation in most framed structures is more cost-effective when plastic ductile hinges are formed in beams at the face of columns [2].

However, it should be emphasized that columns are important members, and how well they function during a seismic event can significantly impact the structure's overall performance [1]. According to Mander et al. [3], having enough transverse reinforcement to confine the concrete, prevent the longitudinal reinforcement from buckling, and avoid brittle shear failure is crucial for the plastic hinge design of reinforced concrete columns.

Numerous analytical and experimental investigations have been carried out so far to look into the impacts of confinement. The potential of interlocking confinement for square RC column was introduced [4] and its behavior under compression was examined [5], and the performance was also further investigated [6]. Sheikh [7] conducted a comparative study on confinement models.

Mander et al. [8] further observed its stress-strain behavior, where its strength and ductility were also examined [9]. The application using high-strength steel bars as confinement was then investigated [10].

Sabariman et al. added steel fiber to reduce the need on transverse steel as confinement [11]. External confinement was also introduced using external collars [12]. It was then simplified into steel bars [13]. FRP straps were also used as confinement [14]. Honestyo et al. [15] used bubble-size plastic straw waste FRP to confined concrete columns. Saafi et al. [16] studied the behavior of concrete columns Confined with FRP tubes. The confinement model for axially loaded concrete confined by circular FRP tubes was also studied [17]. The axial behavior and modeling of confined circular sections using Carbon FRP Jackets were also developed [18]. Lee et al. [19] developed an analytical model for FRP-jacketed concrete columns under axial compression.

Chai et al. [20] proposed an analytical model for steel-jacketed RC columns. Xiao et al. [21] studied the retrofit of RC columns with partially stiffened steel jackets. An experimental investigation of external Confinement of RC Columns using hollow structural section Collars was also proposed [22]. The requirements for confinement in seismic concrete members have also been stipulated in the latest building codes [23]. Sections with square and circular columns were studied in these investigations. The column specimens were subjected to axial and combined axial and bending loadings in both monotonic and cyclic patterns. The volumetric ratio of lateral steel to the concrete core, the yield strength of transverse reinforcement, the ratio of longitudinal steel around the core perimeter, the resulting tie configuration, and the tie spacing were found to be the variables influencing the behavior of confined concrete. There is general agreement that the enhanced stress-strain relationship of confined concrete differs from that of unconfined concrete in three ways: the ultimate compressive strain increases, the post-peak descending branch of the curve is flatter, and compressive strength increases.

Research on confining columns has been expanded for externally applied confinement in addition to the traditional embedded stirrups [12]-[19]. One of the primary reasons it is imperative to develop such a method is the high demand for column retrofits. Experiments have demonstrated the effectiveness of several strategies for retrofitting circular columns [15]-[18], [20]. It is difficult to provide effective confining stress via external retrofit for square and rectangular columns, nevertheless. This issue is found to be addressed by a few experimental and sparsely analytical investigations [19], [21], [22]-[25].

Theoretically, square or rectangular portions retrofitted externally by steel elements (Fig. 1) encounter non-uniform confining stress distribution because of the flexure mechanism of the confining element, which causes the section to bulge under compressive load.

This research examines the advantages of the retrofit strategy through a series of experimental studies on square concrete columns subjected to monotonic axial compressive stress and externally confined by the steel collars. The volumetric ratio of the confining external steel collars is the experiment's primary parameter. The paper also reports the effectiveness of the steel collar in externally confining the existing RC columns under axial compressive loading.

II. Experimental Setup

The objective of leading the compressive test of monotonic-static-axial is to investigate the external steel collar confinement for a conceivable retrofitting method of ineffectively confined concrete columns. Concrete strength with a normal design of $f'_c = 20$ MPa was applied for all concrete specimens. The cross sections were set to be 200×200 mm² and the height was set to be 600 mm.

The cover thickness of the concrete specimens used was 20 mm.

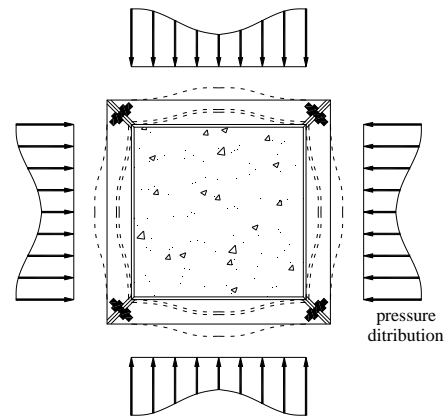


Fig. 1. Non-uniform confining stress of square/rectangular column section externally retrofitted by steel element

The specimens were conditioned to have a middle test region and two non-test regions at both ends of 400 mm and 100 mm, respectively. The column specimens were reinforced with deformed bars for both longitudinal (f_y) and transverse reinforcements (f_{yt}) of 400 MPa. The non-test areas were considered to have denser confinement than the test area. Therefore, no setback was anticipated in these domains. The specimens were then externally confined with the steel angle collars 28 days after casting.

Fig. 2 and Fig. 3 show the illustration of a typical specimen.

II.1. Control Specimens CS-01, CS-02a, CS-03a

As shown in Fig. 4, three specimens, CS-01, CS-02a, and CS-03a, were designated as control specimens. The purpose of these control specimens was to obtain benchmark behaviors under an axial compressive load of normally restricted concrete column specimens.

The construction of CS-01 was conducted without any restriction within the test region. The sole longitudinal reinforcements utilized were 4-D10, which are four 10-mm diameter deformed steel bars. The purpose of the letter "a" behind specimens CS-02 and CS-03 was to indicate that those control specimens' test regions are subject to conventional confinement. The numbers "1" through "4" are written on the specimen's sides in a clockwise manner. The letters "L" and "S" stand for stirrups and strain gauges fastened to longitudinal bars, respectively. The letter L was followed by the two-digit numerals (XY), which were used to indicate that they were fastened to the corners of sides X and Y. In the test region, at side N of the specimens, the two-digit numbers (M-N) that followed the letter S were intended to indicate that they were fastened to stirrups number M (counted from bottom up).

In order to depict the state of columns that did not meet the seismic confinement criteria, CS-02a was created.

Equation (1) only required the transverse reinforcement to meet the minimum shear criterion (ACI 318-19 [23], Section 10.6.2.2):

$$A_{vmin} = \frac{0.062\sqrt{f'_c}b_ws}{f_{yt}} \geq \frac{0.35b_ws}{f_{yt}} \quad (1)$$

where:

- A_{vmin} = minimum area of stirrups (mm²);
- b_w = width of a concrete element (mm);
- f'_c = concrete strength (MPa);
- s = spacing of stirrups (mm);
- f_{yt} = yield strength of stirrups steel (MPa).

Additionally, the confinement spacing that is chosen must not be greater than the maximum shear reinforcement spacing, which is determined by whichever of the following is smaller (ACI 318-19, Section 25.7.2.1): (a) clear spacing of at least (4/3) maximum aggregate size, (b) center-to-center spacing should not exceed the least of 16 diameters of longitudinal bar, 48 diameters of tie bar, and the smallest dimension of member.

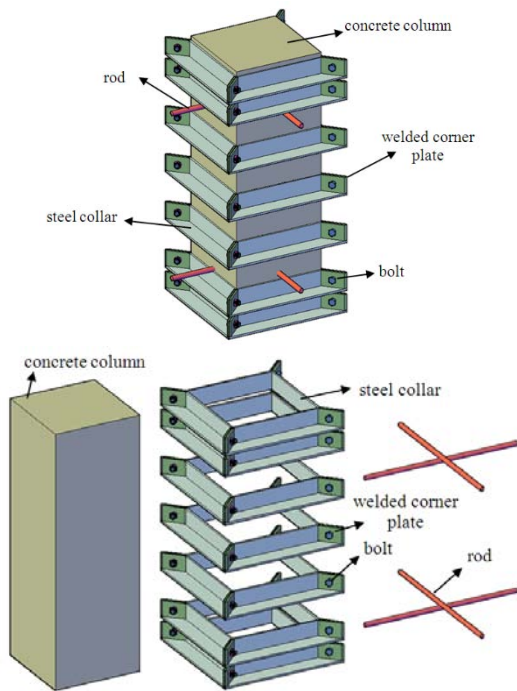


Fig. 2. Three-dimensional and exploded views of the typical specimen

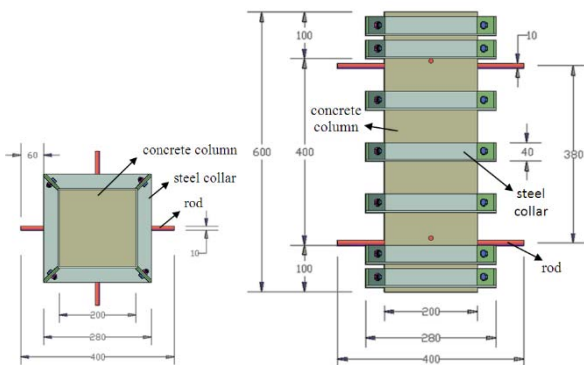


Fig. 3. Top and elevation views of the typical specimen

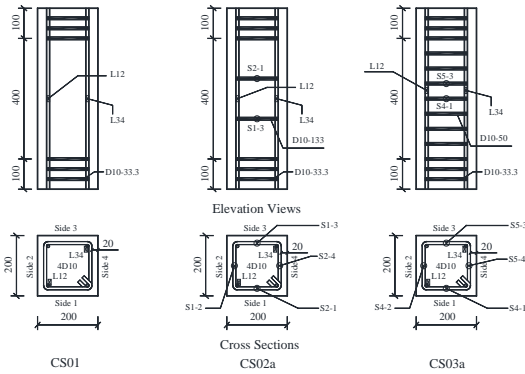


Fig. 4. Specimens CS-01, CS-02a, and CS-03a elevation view and cross-section

The state of columns confined with seismic confinement requirements was intended to be represented by CS-03a. The following are the seismic confinement standards outlined in Table 18.7.5.4 of ACI 318-19:

$$A_{sh1} = 0.09 \left(sb_c \frac{f'_c}{f_{yt}} \right) \quad (2)$$

or:

$$A_{sh2} = 0.3 \left(sb_c \frac{f'_c}{f_{yt}} \right) \left(\frac{A_g}{A_{ch}} - 1 \right) \quad (3)$$

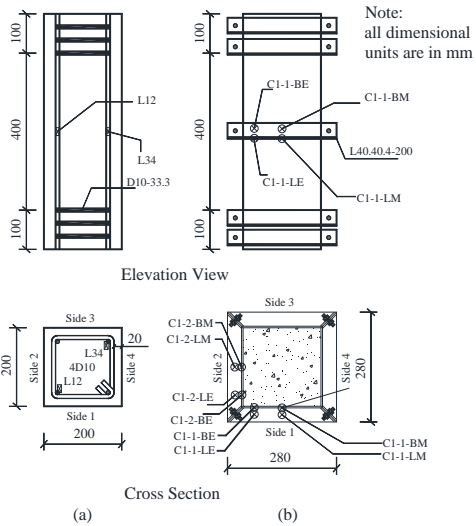
where confined core sectional area, gross cross-sectional area, and confined concrete core dimensions are represented by the variables b_c , A_g , and A_{ch} , respectively.

For a member subject to combined axial and bending pressures in the seismic region defined in ACI 318-19 Section 18.7.5.3, these criteria must be supplemented by the maximum transverse reinforcement spacing, which is the smallest value of these following expressions: one-fourth of the minimum column dimension, for Grade 420 and 550, six and five times diameter of the smallest longitudinal bar, respectively, and $100 \text{ mm} \leq 100 + (350 - h_x)/3 \leq 150 \text{ mm}$, where h_x is the longest distance between the centers of the crossties or hoop legs). For this specimen, confined reinforcing steels of D10–50 were chosen to comply with the requirements.

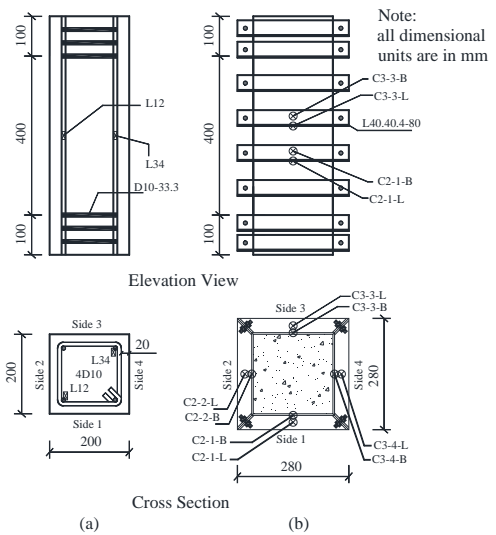
II.2. Specimens S-01, S-02, S-03, S-04, and S-05

S-01 to S-05 were constructed using the same dimensions and specifications as specimen CS-01. Steel angle collars were used to externally confine the specimens inside their test area. The L40 steel angle collars were 40 mm wide and 4 mm thick at the plate.

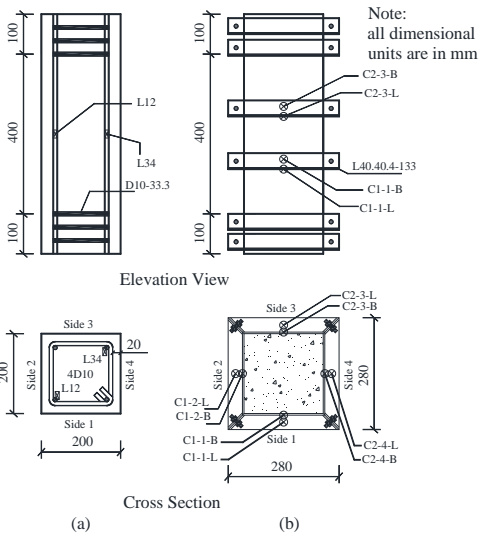
For S-01, S-02, S-03, S-04, and S-05, the volumetric ratios that were intended for these specimens were 3.84, 5.77, 7.68, 9.60, and 11.46 percent, respectively. The examples are displayed in Figs. 5 to 9. Except for the extra coding for strain gauges attached to the steel collars (which began with the letter "C"), every comment in the figure was explained in the same way as the control.



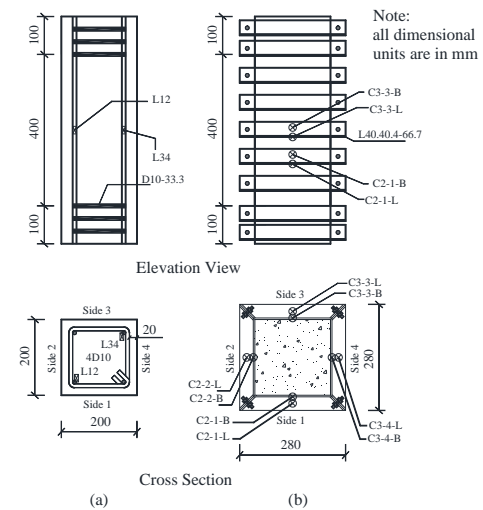
Figs. 5. Elevation view and cross-section of Specimens CS-01, CS-02a, and CS-03a



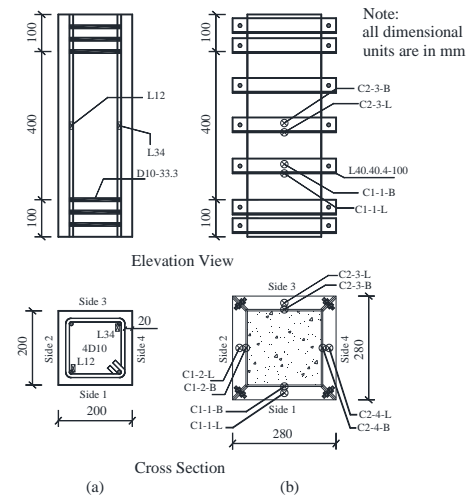
Figs. 8. Elevation view and cross-section of Specimen S-04: (a) reinforcement details; and (b) external steel collars



Figs. 6. Elevation view and cross-section of Specimen S-02: (a) reinforcement details; and (b) external steel collars



Figs. 9. Elevation view and cross-section of Specimen S-05: (a) reinforcement details; and (b) external steel collars

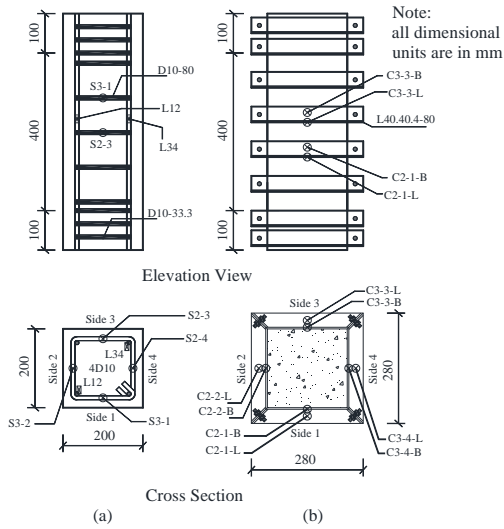


Figs. 7. Elevation view and cross-section of Specimen S-03: (a) reinforcement details; and (b) external steel collars

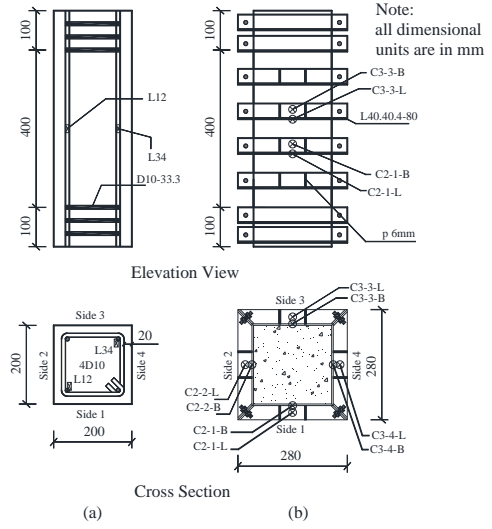
II.3. Variations of Specimen S-04

In order to investigate further the potential of the proposed retrofitting methods, six specimens were built identical to specimen S-04 with certain modifications. The exterior confinement of specimens S-04a and S-04b was identical to that of specimen S-04. They were, nevertheless, confined within their test areas. The purpose of these specimens is to examine the combined impact of both exterior and interior confinements. As seen in Figures 10, specimen S-04a was constructed with the external and interior confinements at the same distances from one another and in the same locations across the test area.

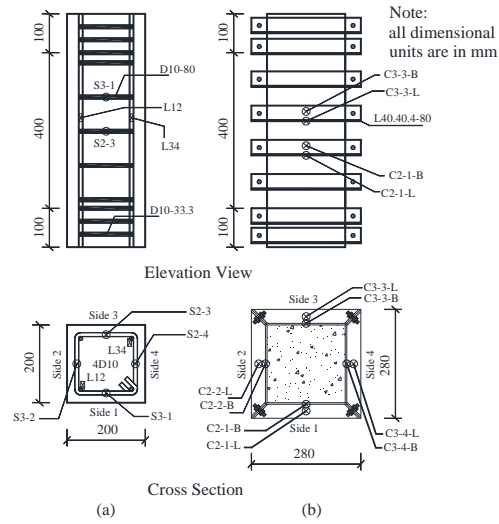
Specimen S-04c and S-04d were created using the same data as specimen S-04, with the exception that a few web stiffeners were included to strengthen the steel angle collars. For specimens S-04c and S-04d, respectively, one and two web stiffeners were put in each steel collar (Figs. 12 and 13).



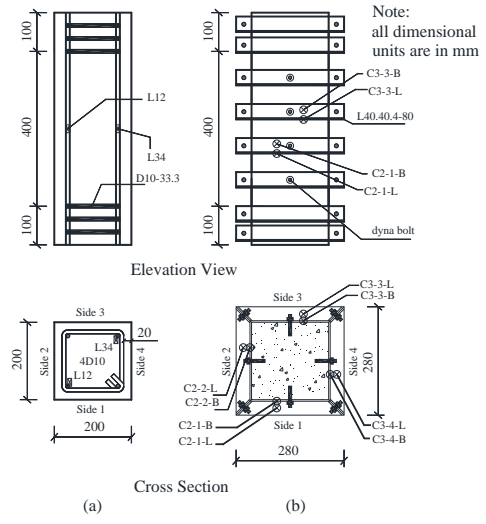
Figs. 10. Elevation view and cross-section of Specimen S-04a: (a) reinforcement details; and (b) external steel collars



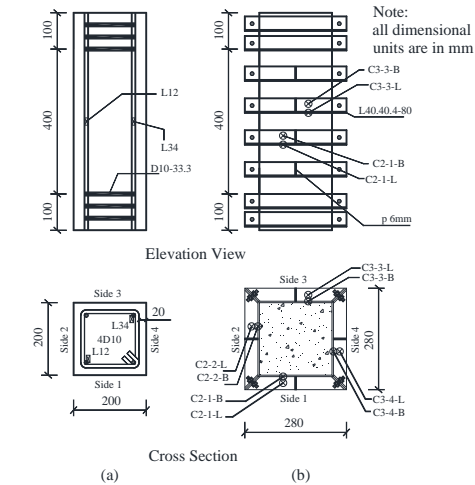
Figs. 13. Elevation view and cross-section of Specimen S-04b: (a) reinforcement details; and (b) external steel collars



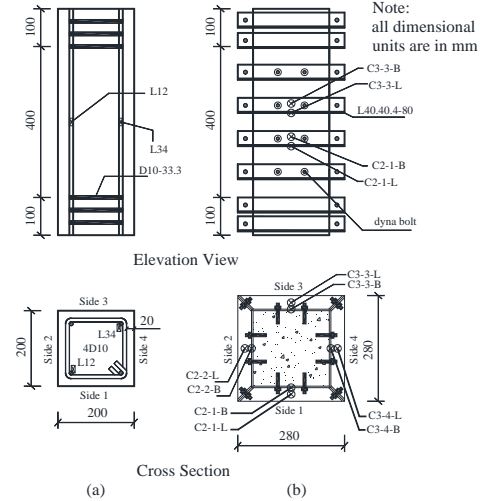
Figs. 11. Elevation view and cross-section of Specimen S-04c: (a) reinforcement details; and (b) external steel collars



Figs. 14. Elevation view and cross-section of Specimen S-04e: (a) reinforcement details; and (b) external steel collars



Figs. 12. Elevation view and cross-section of Specimen S-04c: (a) reinforcement details; and (b) external steel collars



Figs. 15. Elevation view and cross-section of Specimen S-04f: (a) reinforcement details; and (b) external steel collars

III. Experimental Results and Discussion

Table I provides a summary of all specimen details for the axial compressive test. Fig. 16 illustrates the procedure of the test for monotonic axial compressive. During the test, load cells were used to record the columns' axial resistances, and LVDTs were used to measure the axial displacements. If any of the following conditions were met, the testing was terminated: (1) specimen failure; (2) specimen strength resistance dropping under 50%; or (3) measurement constraint of LVDT. The specimens used in this paper are split into two sets to present the results. The standard collared specimens (S-01, S-02, S-03, S-04, S-05) and all control specimens (CS-01, CS-02a, CS-03a) make up the first set. The basic collared specimen S-04 and its variations (S-04a, S-04b, S-04c, S-04d, S-04e, S-04f) make up the second set.

III.1. Specimens CS-01, CS-02a, CS-03a, S-01, S-02, S-03, S-04, and S-05

It was discovered that the specimen of concrete strength was 17.02 MPa from control specimen CS-01. Specimen strain and ultimate strain were 0.23 and 1.37 percent, respectively, and the proportion of the specimen strength to the strength of the cylinder was 0.711.

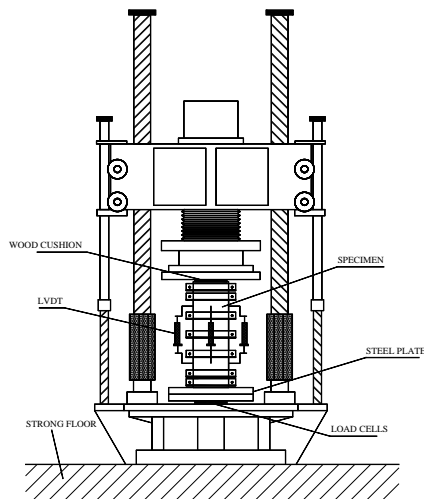


Fig. 16. Typical setup of the monotonic compression test

TABLE I

THE SPECIMEN STRENGTH INCREASE AND FAILURE REMARKS

Specimen ID	Interior confinement	External confinement/steel collars
CS-01	-	-
CS-02a	D.10-133 (VR=0.89%)	-
CS-03a	D.10-50 (VR=2.36%)	-
S-01	-	L40-200 (VR=3.4%)
S-02	-	L40-133 (VR=5.8%)
S-03	-	L40-100 (VR=7.7%)
S-04	-	L40-80 (VR=9.6%)
S-05	-	L40-67 (VR=11.5%)
S-04a	D.10-80 (VR=1.5%)	L40-80 (VR=9.6%)
S-04b	D.10-80 (VR=1.5%)	L40-80 (VR=9.6%)
S-04c	-	L40-80 (VR=9.6%)
S-04d	-	L40-80 (VR=9.6%)
S-04e	-	L40-80 (VR=9.6%)
S-04f	-	L40-80 (VR=9.6%)

Relative strain ductility ratio is a regularly used measure to determine ductility for specimens loaded axially. Table II also includes a list of the ratio of absolute strain ductility. It is evident that CS-01 and CS-02a exhibited extreme behavior of brittle, with strength quickly declining after sampling at 3.27 and 3.01, respectively. With the exception of having a late post-peak ductility response, S-01's behavior was rather similar. Up to the point where it eventually lost strength at roughly 10% axial strain, CS-03a demonstrated good ductility.

Better ductility patterns were seen in collared column specimens with a larger proportion of volumetric, Exception S-04 was the only one to experience a steel collar failure. Specimens S-02, S-03, and S-05 produced ductility values of 4.84, 8.15, and 26.16, respectively, but specimen S-04 only showed 3.46. The suggested retrofitting technique showed that, with the fewest stirrups needed in accordance with seismic regulations, it could achieve performance that was comparable to that of the traditionally restricted specimen CS-03a in terms of strain ductility ratio. Because specific indicators or indications were not shown by the load-displacement curve, it was a little challenging to determine when concrete spalling began during the test. When a specimen exhibits abrupt brittle failure, spalling may start nearly immediately, followed by complete failure. Theoretically, concrete would fracture and spall when it reached the greatest compression strain or about 0.30 percent. It should be mentioned that the axial compression causes significant lateral expansion, not direct tension, which is the origin of this crack. As a result, the first concrete crack found during testing determines the measured strains at spalling.

Because Control Specimen CS-01 failed in a brittle way, the beginning of cracking could not be identified in that instance alone. Initial spalling was observed in the other specimens within the strain around 0.19 to 0.49 %.

The unconfined strength of the concrete specimen is represented by the specimen strength of CS-01. The stress-strain curve is stabilized (all stress values are split by) using this strength in order to examine the impact of confinement on further specimens. Figure 17 displays the contrast of stabilized stress-strain. Table II provides a summary of those specimens' strength growth and failure notes. There has been no discernible increase in strength in CS-02a, which was traditionally limited in the test region with an inadequate volumetric ratio of stirrups. The improved confinement resulted in a strength improvement of 1.206 for the Control specimen CS-03a. There appeared to be a slight deviation in the strength gain of the collared specimens S-02 and S-04. Specimen S-02 revealed a somewhat greater strength enhancement of roughly 1.325, whereas specimen S-04 revealed a somewhat lesser strength gain of roughly 1.232. The predicted strength growth increments for the other collared specimens, S-01, S-03, and S-05, were 1.085, 1.209, and 1.422, respectively. S-03 fared rather well overall, gaining strength in a manner akin to that of CS-03a. On the other hand, S-03 is less ductile than CS-03a. S-05 performed the finest in terms of increases in both ductility and strength.

TABLE II
STRENGTH INCREASES AND REMARK OF SPECIMENS CS-01 TO S-05

Specimen ID	f'_{cc}/f'_{c0}	Remark (descendant branch)
CS-01	1.000	60% of specimen strength was lost following the descendant branch (at strain 0.62%). It was noted that longitudinal bars buckled, and brittle diagonal failure occurred.
CS-02a	0.954	When the descendant branch fell under 50% of its specimen strength at a strain of roughly 1.5%, the test was terminated. Excessive longitudinal bar buckling and damages were noted.
CS-03a	1.206	Because of the LVDT constraint, the test was terminated at 60% specimen strength. Although there was evidence of longitudinal bar buckling, it was still able to withstand the axial force.
S-01	1.085	At around 1.2% strain, strength fell below 50%. It was noted that longitudinal bars buckled, and brittle diagonal failure occurred.
S-02	1.325	At a strain of roughly 3.5%, strength fell below 50% of specimen strength. Longitudinal bar buckling was noted.
S-03	1.209	At a strain of roughly 7.4%, strength fell below 50% of specimen strength. Longitudinal bar buckling was noted.
S-04	1.232	At a strain of roughly 3.8%, strength fell below 50% of its specimen. It was noted that longitudinal bar buckling and Collar 3 failure occurred.
S-05	1.422	Due to damaged Collars 2 and 3, there were two strength drips: one at 74% of specimen strength and the other at 66% of specimen strength.

Two of specimen S-05's steel collars failed throughout the test, although only at comparatively high axial strains of around 8.60 and 11.64%. It was clear from the strain amount that the steel collars and stirrups served as confinement. During the testing, the longitudinal reinforcement was under compression while confinement experienced tension. Only a few representative stress-strain curves for a few specimens are shown in this work because the behaviors are essentially the same and Fig. 18 illustrates the damage. The column failure mechanisms are displayed in Figures 19 and 20. It was clear that the specimen's brittle diagonal failure in CS-01 was caused by the lack of any containment. A brittle failure also befell specimen CS-02a, although the damage was not as great as that of CS-01. Even at extremely high axial strain, specimen CS-03a, which was traditionally contained by stirrups mandated by the seismic requirement, was able to avert a catastrophic brittle fracture of the core.

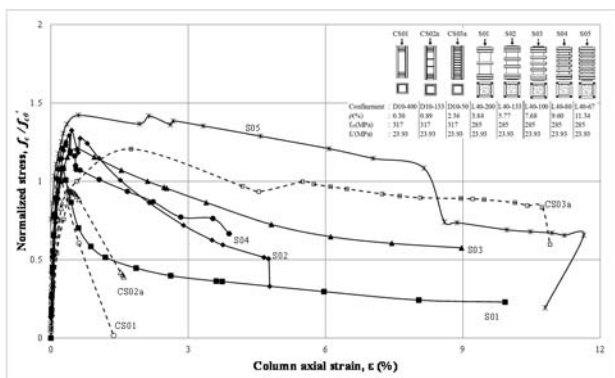


Fig. 17. Control and collared specimens stabilized stress-strain curves

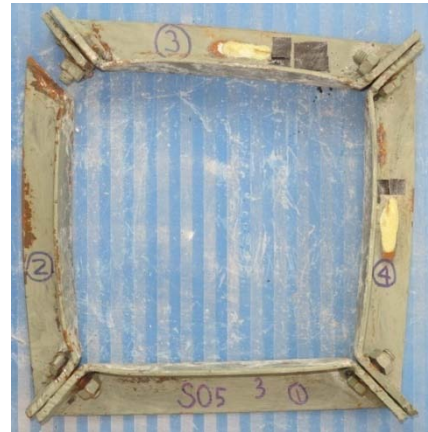


Fig. 18. Collar 3 of S-05 after the accomplishment of the test



Fig. 19. Specimens CS-01, CS-02a, and CS-03a after the accomplishment of the tests



Fig. 20. Specimens S-01, S-02, S-03, S-04, and S-05 after the accomplishment of the tests

The confinement given in specimen S-01 was insufficient for collared specimens, as evidenced by the persistence of the diagonal brittle fracture. Improved steel collar confinement in specimens S-02, S-03, S-04, and S-05 entirely prevented such brittle failure. It is evident that the areas where the steel collars were positioned were protected by concrete. Because of the early breakdown of the weld at one collar's corner, S-04 did not perform as intended. Where the failed steel collar was located, there was significant damage to the concrete. Though it should be highlighted that the damages happened much later in the test, specimen S-05 showed damage that was comparable to that of S-04.

III.2. Specimens CS-01, CS-02a, CS-03a, S-01, S-02, S-03, S-04, and S-05

Six specimens were used in the second round of studies, which were expected to examine the possibilities of steel collar confinement. Figure 21 displays the specimens' stabilized axial stress-strain curves together with the standard specimen S-04. Specimens S-04a and S-04b indicated a highly encouraging finding. The act of the columns without interior stirrups may be greatly enhanced by applying external steel collars, as demonstrated by these two examples, which also demonstrate superior performance.

The strain ductility of S-04a and S-04b was discovered to be 21.21 and 22.58, respectively, while the strength improvements were found to be as high as 1.795 and 1.770, respectively. This suggests that the suggested external confinement method is ideal for RC column retrofitting projects. Despite the two animals' differing interior stirrup locations, their performances were almost

identical. This could be because of the near stirrups and steel collar spacings, making the performance variation less noticeable. For specimens, S-04a and S-04b, the first concrete spalling happened at stresses of roughly 0.34 and 0.25 %, respectively. The specimens S-04c and S-04d exhibited significantly better ductility of 11.08 and 10.47, respectively, and strength enhancements over S-04 (1.286 and 1.365, respectively). Given that the ordinary specimen S-04 experienced collar failure prior and that the strength and ductility increase of S-04c and S-04d performed in between standard specimens S-03 and S-05, it appeared that the web stiffeners' effect was ineffective. The fact that there was no discernible increase in the performance of the specimens enhanced with two web stiffeners (S-04d compared to S-04c) confirmed this. Out of the six specimens, S-04c and S-04d exhibited the least strength enhancements. For specimens S-04c and S-04d, the preliminary spalling happened at strains of approximately 0.50% and 0.30%, respectively. The specimens S-04e and S-04f, whose collar attachments were reinforced with Dyna bolts, exhibited superior ductility and specimen strengths. S-04e and S-04f were found to have ductility values as high as 12.38 and 9.46, respectively. Although S-04e and S-04f's strength improvements (1.588 and 1.562, correspondingly) were higher than those of S-04c and S-04d, they were still less than those of S-04a and S-04b. The performance of the retrofitting procedure has been enhanced by the extra add-on made possible using Dyna bolts. Generally, Figure 22 displays the specimens' bent steel collars following the tests. Figure 23 shows the damage incurred by the specimens. The only specimen with diagonal splitting failure was S-04c. After the testing was completed, the remaining specimens had comparable levels of damage.

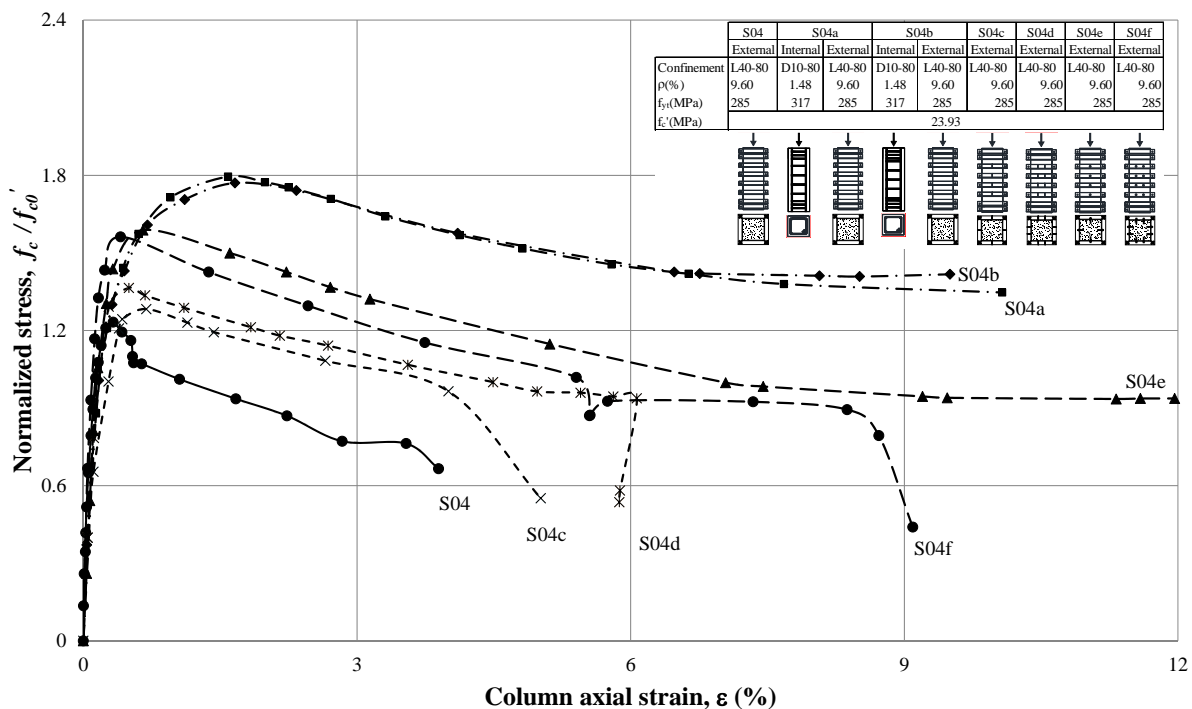


Fig. 21. Specimens S-01, S-02, S-03, S-04, and S-05 after the accomplishment of the tests

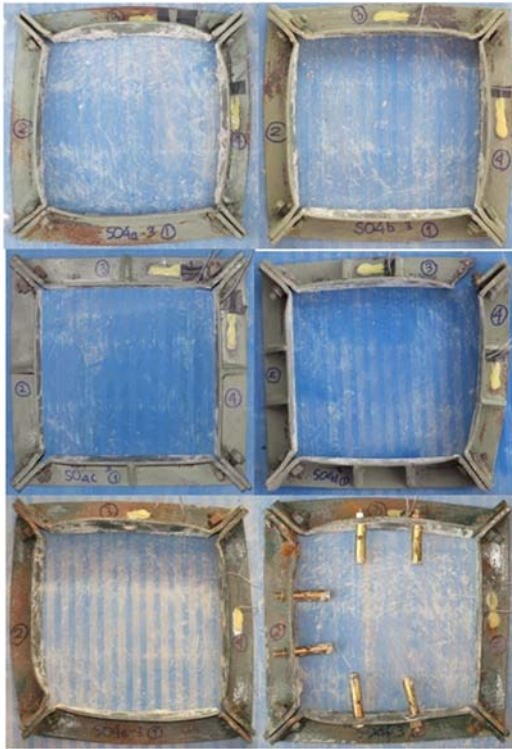


Fig. 22. Steel collars of Specimens S-04a, S-04b, S-04c, S-04d, S-04e, and S-04f after accomplishment of the tests



Fig. 23. Specimens S-04a, S-04b, S-04c, S-04d, S-04e, and S-04f after accomplishment of the tests

In order to evaluate the effectiveness of the suggested external confining technique, a series of fourteen column specimens was subjected to monotonic compressive loads.

Several inferences can be drawn from the experimental

data, including the following: specimens externally constrained using the suggested method show improved axial stress-strain behavior in comparison to the plain concrete; A brittle failure was detected in specimens with fewer steel collars, while ductile behaviors were noted in specimens with more steel collars; It is evident by looking at the damaged patterns that the steel collars serve as the limiting factor. Compared to other areas, the concrete strips covered by the steel collars that restricted movement displayed less damage; Control behaves similarly in specimens CS-03a and S-03, which have three steel collars with a 7.68% volumetric proportion. the specimen with five steel collars, S-05, which was the most heavily confined and had a volumetric proportion of confining component of 11.46 percent, could range up to 1.422 times the strength of CS-01 and show more than 10.00%; The specimens S-04c and S-04d, which were contained by steel collars that had been changed and included web stiffeners, showed a very slight improvement in their performances (in terms of increased strength and ductility). The impact of the web stiffeners was not noticeable since the failure mechanism of the short-length steel angle section was not guided by any local instability of the sections; Through specimens S-04a and S-04b, it was shown that the combination of external steel collars and traditional interior stirrups significantly improved strength and ductility. The specimens that showed the greatest improvement were those with stronger steel collars (by adding bolts for extra add-on sites), and specimens S-04e and S-04f (with a volumetric proportion of 9.6%). Both specimens exhibited specimen strengths that were nearly 1.8 times larger than those of CS-01, with highly significant axial deformability.

IV. Conclusion

A proposed analytical model can be used to predict the external steel collars' confined square concrete columns specimen stresses. The main objective of the research is to derive the corresponding effective uniform confining stress (f_{le}) that external confining steel collars can serve in the model. A series of column specimens was constructed and subjected to a constant compression load. The test results demonstrated a strong linear relationship between the normalized equivalent effective uniform confining stress (f_{cc}/f_{c0}) and the normalized confined specimen strength (f_{le}/f_{c0}). Additionally, a design retrofitting strategy is recommended. The influence of combined interior and external confinements in the analytical approach is proposed. To validate the retrofit methodology, the RC column specimens were also investigated. The outcome demonstrated a very good performance contributed by the steel collars. More RC column specimens are required to further validate the application of the proposed model in predicting more varieties such various sizes of columns and steel collars, application of various sizes and numbers of stiffeners and dynabolts, etc.

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