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Stress-Strain Model of Concrete Columns Externally Collared with Steel Angles Subjected to Axial Compression

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Abstract – *RC structures with poorly confined columns have been known to have poor performance during earthquake events. Brittle failures of columns with low ductility can lead to progressive overall building collapses. Many confining techniques have been proven to be successful in retrofitting circular columns. However, for square or rectangular columns, providing effective confining stress by the external retrofitting method is not a simple task due to the high stress concentration occurring at the column's corners. In this paper, an analytical model for the axial stress-strain relationship of square concrete columns confined only by external steel collars has been developed. The combined effect of conventional internal stirrups and the external steel collars on the improved stress-strain relationship has been also developed analytically and proposed as the retrofit design approach. Comparison with experimental results has shown that the axial stress-strain relationship of square RC columns could be well predicted by the proposed retrofit design approach.*

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

Nomenclature

$P_{\max}$	Specimen's maximum axial resistance
$P_{c\max}$	Concrete maximum axial resistance
$\varepsilon_{P\max}$	Axial strain consistent to $P_{\max}$
ε_{cc}	Axial strain consistent to $P_{c\max}$
f'_{cc}	Confined concrete compressive strength [MPa]
ε'_{cc}	Confined concrete compressive strain
ε'_{c0}	Unconfined concrete compressive strain
E_{sec}	Confined concrete modulus of elasticity [MPa]
r	Constant
E_c	Plain concrete modulus of elasticity [MPa]
ε_c	Strain of concrete

I. Introduction

As seismic activity has been better understood, codes have become more demanding, necessitating the reinforcing and retrofitting of many old Reinforced Concrete (RC) structures. Liu et al. [1] have stated that the following deficiencies typically result from old, existing RC frame structures. A weak column can result in a mechanism of strong-column and weak-beam. Insufficient lap splices directly above floor levels can cause inelastic movements to concentrate with the higher flexural need.

Inadequate transverse reinforcement can confine the center of the column and prevent the longitudinal reinforcement buckling. Inadequate shear strength can develop the column's flexural capacity, or it can potentially degrade with increasing flexural ductility demand. Lastly, insufficient dimensions and detailings at the beam-to-column joint can cause problems. 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 not in columns [2].

However, a failure of the single column can cause further failures and possibly result in the collapse of the entire building. Any structural building system would have columns as important members, and the building's ability to survive a seismic event is greatly impacted by how well it performs [1]. According to Mander et al. [3], having enough transverse reinforcement to confine the concrete, avoid the longitudinal reinforcement from buckling, and prevent brittle shear failure is crucial for the plastic hinge design of reinforced concrete columns.

Strength and ductility in concrete columns have been found to be improved by transverse confining stress [4], [5]. Numerous analytical models have been carried out to date to look at the impacts of confinement [6]-[9].

Typically, traditional transverse reinforcement (stirrups) provides confining stress in columns. Different methods to improve the performance of concrete, such as the use of high-strength steel bars as confinement, have been introduced [10]. The application of confinement has been extended as an interlocking system of multiple confinement [11], [12]. A combination with steel fiber has been also investigated to seek further the effectiveness of confinement [13]. Sections with square and circular columns have been studied in these investigations. The specimens have been subjected to axial and combined axial and bending loadings in both monotonic and cyclic patterns. Analytical models have only predicted the envelope curves in the situation of cyclic loading. The elements that have been found to be affecting the behavior of confined concrete have included the volumetric proportion of side steel to the concrete central, the transverse reinforcement yield stress, the longitudinal steel proportion around the core perimeter, the resulting tie design, and the spacing tie. There is consensus regarding the differences between the enhanced relationship of stress-strain of confined concrete and that of unconfined concrete. These differences include the enhancement of compressive strength, the curve flatter post-peak descending branch, and the ultimate compressive strain increase (an increase in ductility). This has generally enhanced relationship between stress and strain in confined concrete is seen in Fig. 1. Here, $f_t, f'_{c0}, f'_{cc}, \varepsilon_t, \varepsilon_{c0}, \varepsilon_{cc}, \varepsilon_{cu}, \varepsilon_{sp}, E_c$ and s_{sc} represent its concrete's tensile strength, its compressive strength in an unconfined condition, its compressive strength in a confined condition, its rupture tensile strain, the concrete strains corresponding to the unconfined concrete's peak strength, the concrete strains corresponding to the confined concrete's peak strength, the ultimate compressive strain of the confined concrete, the strain at which the concrete cover is assumed completely spalled, the concrete's modulus of elasticity, and the secant modulus of confined concrete at peak stress, respectively.

Investigation on confined columns has recently been expanded for confinement that is applied externally, such as steel jacketing [14], Fiber-Reinforced Polymer (FRP) tubes [15], [16], and Fiber-Reinforced Polymer (FRP) jackets [17]-[19] for external confinement of concrete piers or columns. In order to make economic applications of FRP, GFRP straps have been introduced as confinement for concrete columns [20]. Other methods have been to implement light steel collars to improve the performance of concrete columns as a confining system [21], [22]. One of the primary reasons it is imperative that such an approach be created is the high demand for column retrofits. Experiments have demonstrated the effectiveness of several strategies for retrofitting circular columns [14]-[17]. It is difficult to provide effective confining stress via external retrofit for square and rectangular columns, nevertheless. This issue is found out to be addressed by a few experimental and sparsely analytical investigations [18]-[22].

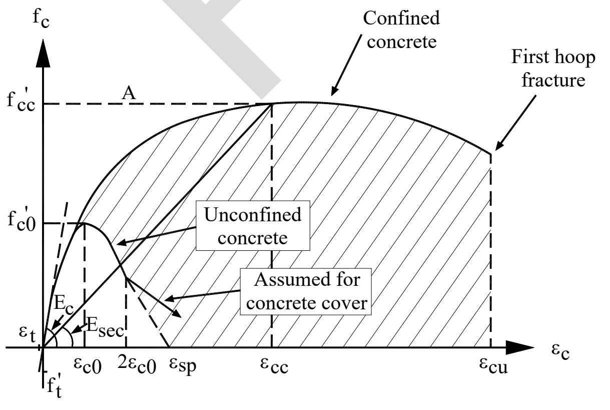


Fig. 1. Typically improved stress-strain curve of confined concrete (modified from [3])

The external steel collars confined square parts (Fig. 3) similarly experience the distribution of non-uniform stress (Fig. 2), which is like typical internal confinement. In addition to this comparable circumstance, the column behavior becomes more difficult when using the external confinement strategy. Compared to traditional stirrups, the mechanism of failure and behavior of contact between concrete and the element of external confinement may change. This paper presents an introduction and a discussion on the analytical approach. The work elaborates a suggested mathematical model for the curve prediction of axial stress-strain of external steel collars confined square columns. As a retrofit design strategy, the combined impact of external steel collars and traditional internal stirrups on the enhanced stress-strain relationship is also analytically suggested. From the investigation, it can be found out that the introduction of external steel collars has enhanced the performance of concrete columns. It can also be concluded that the more collars are used, the better the performance of the column is.

II. Proposed Analytical Model

One of the most crucial aspects of modeling the axial stress-strain relationship of confined concrete analytically is predicting the transverse confining stress.

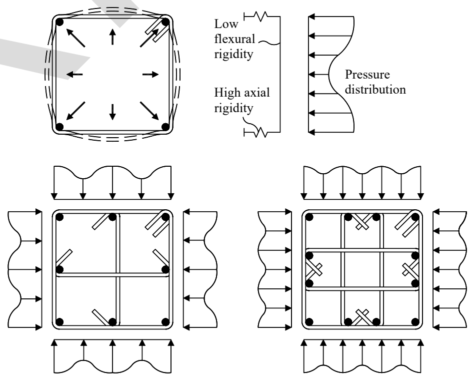


Fig. 2. Rectangular stirrups confined square column non-uniform confining stress (modified from [5])

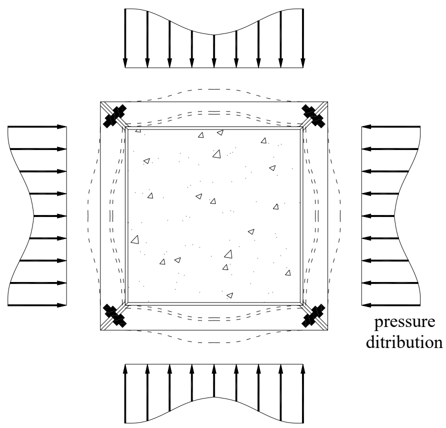


Fig. 3. Scheme of externally steel collars-confined square column non-uniform confining stress

Prior studies [3]-[6] have suggested confining stress models that are supplied on square, rectangular, and circular sections by internal confinement elements (ties and spirals). A retrofit design strategy for reinforced concrete columns with partly stiffened steel jackets has been suggested recently by Xiao and Wu [18]. The methods from earlier studies [3]-[6], [18] have been used in this investigation, with modifications made as needed for the external steel collar retrofitting methodology. The following parts provide an explanation of the model's evolution.

II.1. Main Effective Equivalent Uniform Confining Stress Provided by Steel Collars (f_{le})

A modified square concrete column with steel angle sections of steel collars is shown in Fig. 4. The notations b, s_{sc}, and s_{ssc} in the diagram stand for the concrete width, the steel collar spacing, and the clear steel collar spacing, respectively. As illustrated in Fig. 3, the stress of confining is ununiform in a column cross-section across the confinement device. Across the column section, this condition results in certain regions that are ineffectively contained. An ineffectively confined region $b^2/6$ is produced for each parabola in Fig. 5(a) when the arced action is supposed to behave as a 45-degree initial tangent slope at the corners of second-degree parabolas [3], [5].

Equation (1) can be used to compute an ineffectively confined area expression (A_{par}), which would account for the region of parabolic or the column section side:

$$A_{par} = \frac{2}{3} b^2 \tag{1}$$

As shown in Fig. 5(b), vertically positioned between neighboring confinement components are assumed to be the parabolic sections of ineffectively confined. Equation (2) gives the average effectively-confined cross-sectional area (A_e) of Mander et al. [3] after considering both of vertical and horizontal directions of ineffective zones:

$$A_e = A_c\left(1-\frac{A_{par}}{A_c}\right)\left(1-\frac{s_{scc}}{2b}\right)^2 \tag{2}$$

where, in the case of externally restricted columns, A_c is the concrete core area, or the gross cross-sectional area of the column ($=b^2$). Equation (3) proposes a confinement efficacy factor (k_e), which is further discussed:

$$k_e = \frac{A_e}{A_{cc}} \tag{3}$$

where, A_{cc} is the columns' net core area (which is A_{cc} minus the size of any longitudinal bars). Equation (4) and Fig. 6 provide the uniform confining pressure of effective equivalent (f_{le}), which is modified by adding k_e to account for the actual non-uniform lateral confining pressure.

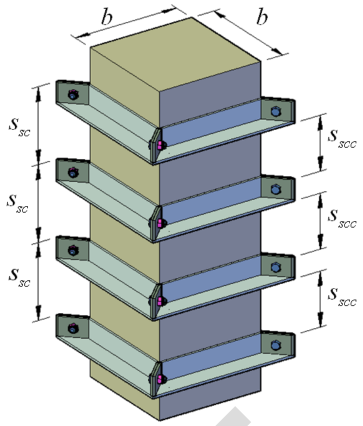
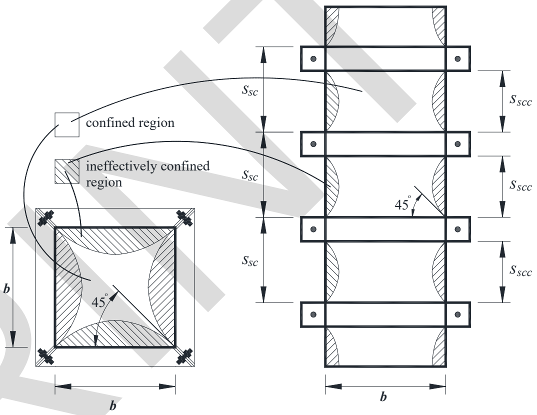


Fig. 4. Illustration of typical three-dimensional column specimens



(a) (b)

Figs. 5. The region of ineffectively confined concrete at (a) cross-section; and (b) the column height

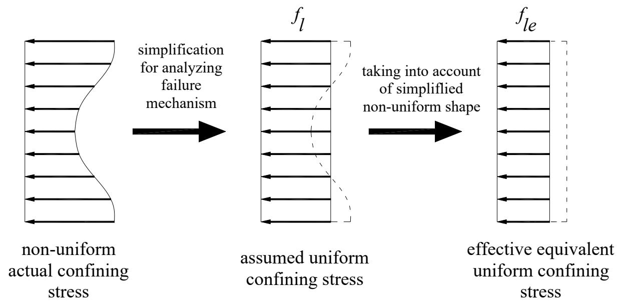


Fig. 6. Uniform confining stress of effective equivalent

The modification of f_l, which will be discussed later, results in the confining pressure of equivalent uniform (f_{le}):

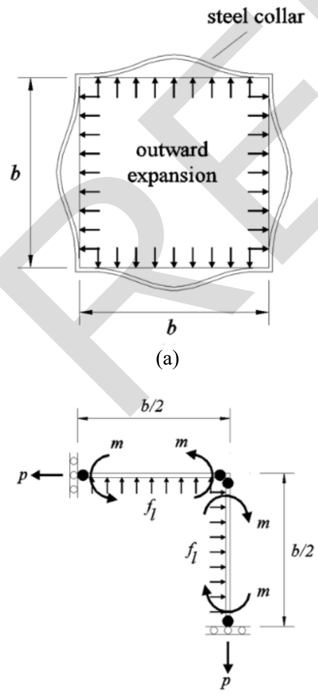
$$f_{le} = k_e f_l \tag{4}$$

II.2. Confining Pressure Provided by Steel Collars (f_l)

Externally confined rectangular concrete columns frequently have confinement that is more efficient at the corners because of the stronger regions of the confinement elements [25], [26]. A comparable situation is also shown by conservatively confined concrete with inside transverse reinforcement. Xiao and Wu [18] have claimed that external steel collars use an axial and combination bending mechanism to provide the confining pressure. Because of the comparatively moderate bending stiffness, this process differs from the statement for reinforcement of transverse, which solely relies on the action of the axial. First, Fig. 7(a) shows a concrete column with an externally retrofitted bulge under axial compressive stress. It is believed that the steel collars will flex in a way that preserves the concrete's ability to expand outward. It makes sense that the mid-sides would have more distortion than the corners. In the middle of the columns and at their corners, it is expected that the steel collars will break due to a combination of axial and bending mechanisms (plastic hinges are designed). By assuming uniformly generated confining pressure, simplified is the real non-uniform confining pressure generated. The forces in stability (the double symmetric condition only allows for analysis of 25% of the model) along the plane of cross-sectional are depicted in Fig. 7(b). Equations (5) and (6) allow for the expression of the axial p and bending moment m created in the steel collars as a function of the equal uniform confining pressure f_l, the breadth of the column section b, and the space of the steel collars s_{sc}. This is achieved by using force equilibrium.

$$p = f_l \frac{b}{2} s_{sc} \tag{5}$$

$$m = f_l \frac{b^2}{16} s_{sc} \tag{6}$$



Figs. 7. (a) Lateral expansion of concrete column and (b) Forces analyzed equilibrium

By using the combination of the steel collar's criterion for axial and bending failure (Eqs. (7)) from the Indonesian Structural Steel Code [23], it is possible to determine the steel collar's nominal capacities of axial and bending (p_n and m_n). Reduction factors (ϕ) should be assumed to be 1.0 for determining the nominal capacity:

$$\frac{p}{\phi p_n} + \frac{8}{9}\frac{m}{\phi m_n} = 1.0 \quad \text{for} \quad \frac{p}{\phi p_n} \geq 0.2 \tag{7-a}$$

$$\frac{p}{2\phi p_n} + \frac{m}{\phi m_n} = 1.0 \quad \text{for} \quad \frac{p}{\phi p_n} < 0.2 \tag{7-b}$$

II.3. Peak Strength of Confined Concrete $\left(f'_{cc}\right)$

All the specimens had their cross sections and heights fixed at 200 × 200 mm^2 and 600 mm, respectively. The employed clear concrete cover had a thickness of 20 mm.

The specimens have been configured with two 100 mm non-test portions at either end or a 400 mm intermediate test area. A 20 mm clear concrete cover has been employed. For all test specimens, normal concrete strength $\left(f'_c = 20 \text{ MPa}\right)$ has been utilized. Deformed bars have been used to strengthen the specimens for both transverse (f_y= 400 MPa) and longitudinal (f_{yt} = 400 MPa) purposes. Denser confinement than that of the test section has been intended for the non-test portions. Then the specimens were externally confined by the steel angle collars at 28 days post-casting. Four specimens with varying degrees of confinement have been designed while maintaining this configuration $\left(f'_{c0}\right)$. Initially, the specimen CS01 (Figure 8) was built without any confinement inside the test area, and the only longitudinal reinforcements utilized were 4-D10, or four 10-mm diameter deformed steel bars. The letters CS have been the first two digits of the specimen ID, then a number. The terms "Control" and "Specimen" are denoted by the letters "C" and "S," respectively. The last numbers are just the specimens' consecutive numbering, and the number "0" has denoted the monotonic axial compressive loading test.

The purpose of this specimen has been to provide the standard strength of untreated concrete $\left(f'_{c0}\right)$. Second, three representative specimens with the same dimensions and characteristics as Specimen CS01 have been further developed into Specimens S01, S03, and S05 (Figures 9).

Steel angle collars have been used to confine externally the specimens inside their test area. Steel collars constructed from steel angle sections have served as the external confinement elements (L40.40.4). For S01, S03, and S05, the volumetric ratios that have been intended for these specimens have been 3.84, 7.68, and 11.46 percent, respectively. The Structural Laboratory of the Research Center for Human Settlement, Ministry of Public Works, Indonesia, has been the site of the experimental tests. In order to find out the mechanical characteristics of steel bars and steel angle collars, tension tests have been carried out.

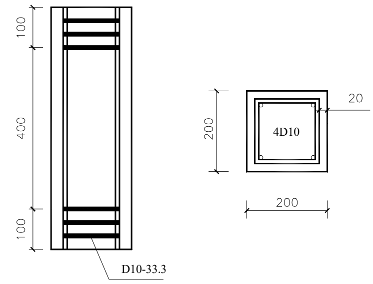
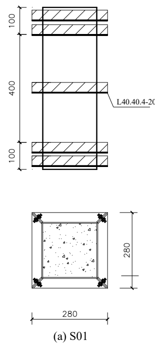
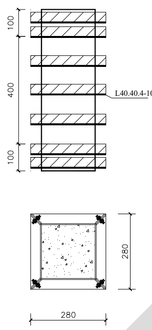


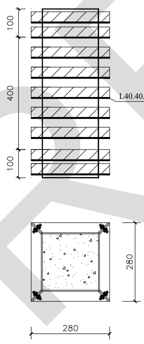
Fig. 8. Elevation and plan views of control specimen CS01



(a) S01



(b) S03



(c) S05

Figs. 9. Elevation and plan views of collared specimens: (a) S01; (b) S03; and (c) S05

With three samples with a nominal diameter of 9.5 mm, the specimen's deformed bars (D10) had an average yield strength (f_y) of 317 MPa with a standard deviation of 5.9 MPa. The equivalent standard deviation for the mean tensile strength (486 MPa) has been 3.8 MPa. A strip plate cut from the steel angle section has undergone a tension test, and the yield strength (f_{ysc}) has been found to be 285 MPa. Figure 10 shows how the compression test is set up.

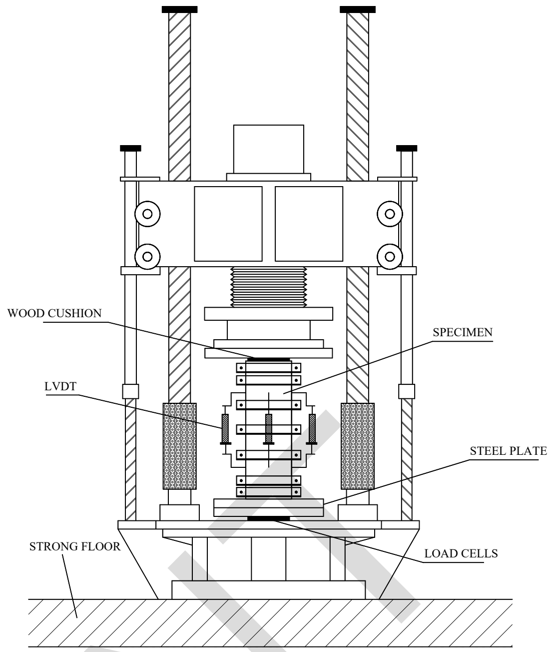


Fig. 10. Typical setup of the monotonic compression test

For the compression testing, four LVDTs have been utilized to measure the axial displacement and a load cell to measure the vertical load. A 5000 kN universal testing apparatus has been used to test each specimen. The specimens have undergone axial concentric loading that has been gradually increased. If any of the following conditions have been satisfied, the testing has been stopped: (1) specimen failure; (2) axial resistance falling below 50% of peak strength; or (3) LVDT capacity constraint. Important information from each specimen is enumerated in Table I. It has been discovered that the concrete strength has been 17.02 MPa from control specimen CS01. The unconfined strength of the concrete specimen is represented by the peak strength of CS01 $\left(f'_{c0}\right)$. The stress-strain curve is normalized (all stress values are split by) by using this strength in order to examine the impact of confinement on further specimens.

Figure 11 displays the initial set of tests' normalized stress-strain comparison.

Table II summarizes the strength increase and the failure notes. The predicted strength growth increments for the other collared specimens, S01, S03, and S05, have been 1.085, 1.209, and 1.422, respectively. By using regression analysis, the observed formula to predict the peak strength has been obtained.

TABLE I
COMPRESSION TEST RESULTS

Parameters	CS01	S01	S03	S05
P_{max} (kN)	762.9	822.9	906.9	1051.0
P_{cmax} (kN)	675.8	733.0	817.1	961.2
$\varepsilon_{P\max}$ (%)	0.23	0.26	0.57	1.83
ε_{cc} (%)	0.23	0.26	0.57	1.83

Notes: $P_{\max}$ is the specimen's maximum axial resistance, $P_{c\max}$ is the concrete maximum axial resistance $\varepsilon_{P\max}$ is the axial strain corresponding to $P_{\max}$, ε_{cc} is the axial strain corresponding to $P_{c\max}$

The normalized effective equivalent uniform confining pressure $\left(f_{le}/f'_{c0}\right)$ and the normalized peak strength $\left(f'_{cc}/f'_{c0}\right)$ have been compared by using regression analysis. As seen in Figure 12, it has been discovered that the linear equation (correlation coefficient, R=0.992) has accurately represented the relationship between the two parameters. Equation (8), which has been derived empirically, can be used to compute the peak strength once the effective equivalent uniform confining pressure has been established:

$$f'_{cc} = f'_{c0}\left(1.0173 + 5.7558\frac{f_{le}}{f'_{c0}}\right) \tag{8}$$

TABLE II
THE SPECIMEN STRENGTH INCREASE AND FAILURE REMARKS

Specimen ID	f'_{cc}/f'_{c0}	Remark for descending brunch
CS01	1.000	60% of peak strength was lost following the descending branch (at strain 0.62%). It was noted that longitudinal bars buckled, and brittle diagonal failure occurred.
S01	1.085	At around 1.2% strain, strength fell below 50%. It was noted that longitudinal bars buckled, and brittle diagonal failure occurred.
S03	1.299	At a strain of roughly 7.4%, strength fell below 50% of peak strength. Longitudinal bar buckling was noted.
S05	1.422	There were two strength reductions because of broken Collars 2 and 3, which occurred at 74% of peak strength (strain 8.60%) and 66% of peak strength (strain 11.64%), respectively. It was also noted that longitudinal bars buckled.

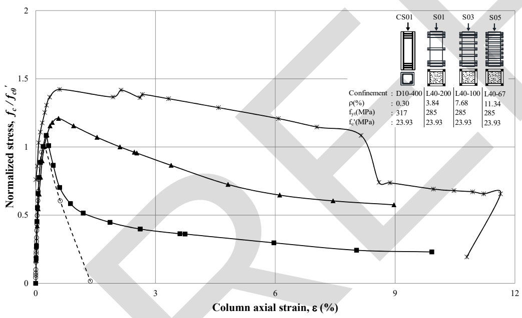


Fig. 11. The specimens' normalized axial stress-strain relationships

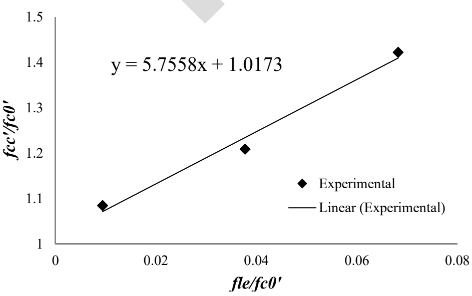


Fig. 12. Peak strength and effective uniform confining pressure linear relationship

II.4. Stress-Strain Relationship

Equations (9)-(12) show how the remainder of the model can adopt the Mander model [3] given the peak power provided by Equation (8):

$$\varepsilon'_{cc} = \varepsilon'_{c0}\left(1 + 5\left(\frac{f'_{cc}}{f'_{c0}} - 1\right)\right) \tag{9}$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon'_{cc}} \tag{10}$$

$$r = \frac{E_c}{E_c - E_{sec}} \tag{11}$$

$$f_c\left(\varepsilon_c\right) = \frac{f'_{cc}\left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right)r}{r - 1 + \left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right)^r} \tag{12}$$

where f'_{cc} is the confined concrete compressive strength (MPa), ε'_{cc} is the confined concrete compressive strain corresponding to f'_{cc}, ε'_{c0} is the unconfined concrete compressive strain corresponding to f'_{c0}, E_{sec} is the confined concrete modulus elasticity (MPa), r is a constant, E_c is the plain concrete modulus of elasticity (MPa), $f_c(\varepsilon_c)$ is the concrete stress as a function of concrete strain (MPa), ε_c is the concrete strain. Figure 13 shows comparisons between the experimental data (S01, S03, and S05) and their analytical predictions. The suggested analytical models in the figure are indicated by the captions of the curves that end in "-Prop." It can be noted that the suggested analytical model can predict the normalized experimental stress-strain curves of Specimen S03 very well. The suggested model significantly overestimates the prediction for S01, while slightly underestimating the prediction for S05 prior to the first collar failure at a rather high axial strain value.

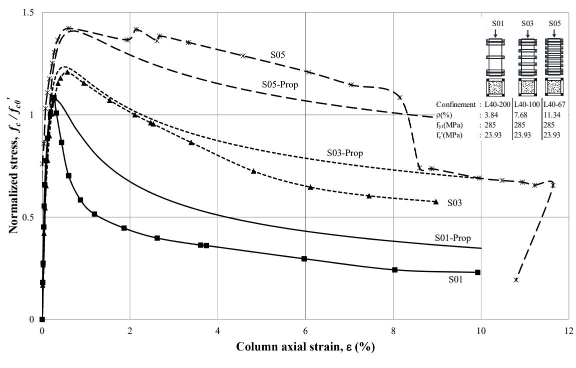


Fig. 13. Proposed analytical model normalized stress vs. axial strain and experimental results of S01, S03, and S05

II.5. Proposed Design Procedure for Strengthening Deficient Square RC Columns with External Steel Collar Retrofit

This segment presents a technique of design for retrofitting using the suggested exterior retrofit method.

Equations (13) and (14) [18] indicate the minimum confinement required of RC columns in the seismic area ensuring the rotational deformability of the possible plastic hinges near the column ends:

$$A_{sh1} = 0.09\left(sb_c \frac{f_c^{'}}{f_{yh}} \right) \tag{13}$$

$$A_{sh2} = 0.3\left(sb_c \frac{f_c^{'}}{f_{yh}} \right)\left(\frac{A_g}{A_{ch}} - 1 \right) \tag{14}$$

The underlying idea behind those statements is that constrained columns ought to keep their axial capacities even after the concrete cover spalls. In order to put it briefly, confinement needs to be set up so that the strength gained in the concrete core and the concrete cover are equal [24]. It is recommended that a column under consideration is given the corresponding confinement pressure for retrofit design [18]. Since the joint axial and the bending mechanisms of the steel collar parts are involved in this suggested retrofitting approach, achieving the desired equivalent confinement pressure is a difficult challenge (Equation (7)).

Additionally, as the method of retrofitting is typically applied to already existing inadequately restricted columns that already have traditional stirrups, the mathematical formulations will be more complicated. The existence of two coupled forms of exterior and inside confinement is the primary problem.

An algorithm to determine the combined effects of exterior and interior confinements is devised in this study.

The technique can be applied with the use of a computer, and the design of retrofit work can be easily simulated to meet a specific confinement target (such as the confinement mandated by a code). The basic concept is to superimpose the extra exterior confinement on top of the existing interior confinement effects. Assumed to be A_{ei} and A_{ee}, respectively, the effectively limited areas impacted by the internal and external confinements are shown in Figure 14.

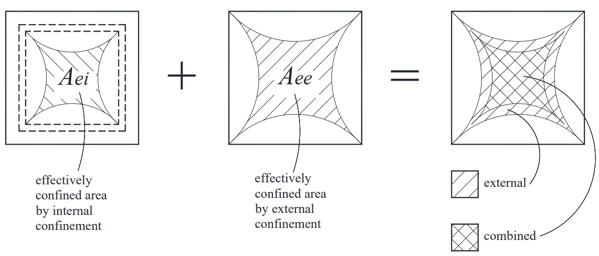


Fig. 14. Effectively Confined Area due to the internal and external confinement

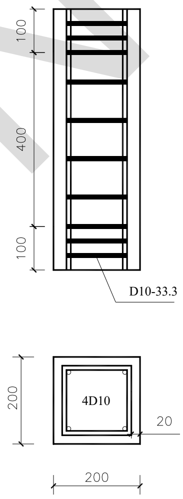
While the zone A_{ee}-A_{ei} solely feels the confining pressure from exterior confinement, the area A_{ei} feels the pressure from both internal and external confinements.

Equation (8) explains how to derive the confining pressure from the exterior confinement (f_{le}) by using the proposed analytical model, whereas the classic model presented by Mander et al. [3] can be used to determine the confining pressure from the interior confinement (f_{li}).

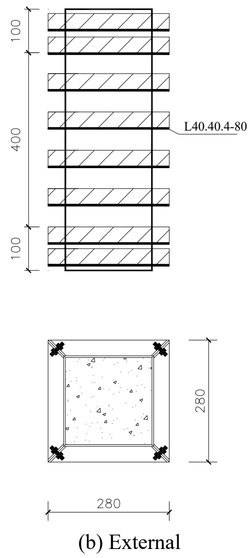
In order to syndicate these effects, it is planned to implement the average confining pressure (f_{lcomb}), which is relational to their affected zones as defined in Equation (15).

This confining pressure average can be used along with the proposed analytical model (Equations (9)-(12)) to express the axial stress-strain relationship because of both internal and external confinements:

$$f_{lcomb} = \frac{\left(f_{li} + f_{le} \right) A_{ei} + f_{le}\left(A_{ee} - A_{ei} \right)}{A_{ee}} \tag{15}$$



(a) Internal



(b) External

Figs. 15. Elevation view and cross-section of Specimens S04a: (a) reinforcement detail; and (b) external steel collars

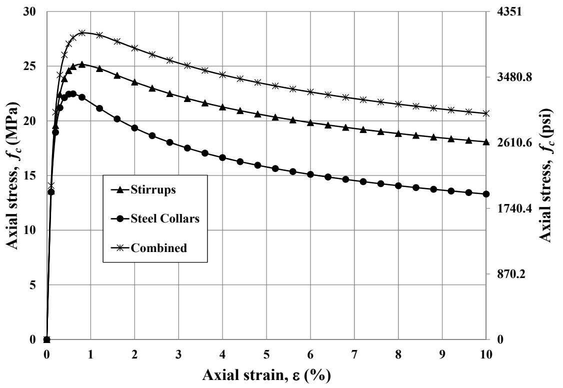


Fig. 16. Stress-strain relationship of S04a (retrofit design approach)

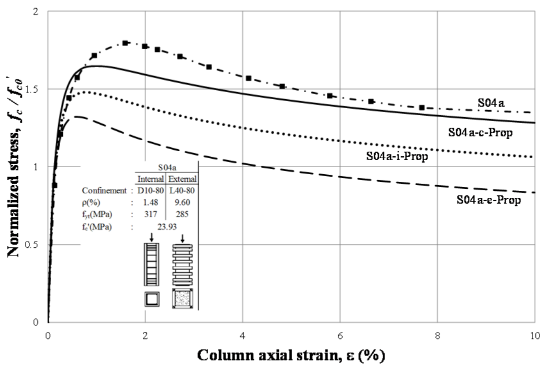


Fig. 17. Normalized stress vs axial strain of combined effect of internal and external confinement and experimental results of Specimen S04a

An additional specimen (S04a) has been constructed and tested by using the same experimental configuration to confirm the efficiency of the suggested design method.

The interior and the external confinement locations of the specimens have been arranged to coincide with one another. The example is shown in Figures 15.

III. Discussion on Results

Figure 16 displays both the individual and the combined effects of internal and exterior confinement as well as the analytical stress-strain relationship curves. The normalized stress-strain relationships of experimental specimen S04a are compared with the suggested retrofit design model in Figure 17.

The outcome demonstrates that the analytical model has a reasonable ability to predict the observed behavior. It is discovered that the suggested method's results understate experimental data, yet this is still acceptable because it indicates that the design is safe and acceptable.

The suggested analytical model curve legends are designated as "S04ab-x-Prop." The impacts of the interior, exterior, and combined confinements for "i," "e," and "c" are shown by the index "x," respectively.

IV. Conclusion

A proposed analytical model can be used to forecast the external steel collar's confined square concrete column's peak stress. The primary suggestion of the model is to derive the corresponding effective uniform confining stress (f_{le}) that external steel confining materials provide.

A collection of specimens was constructed and subjected to a constant compression load. The test's results have demonstrated a strong linear relationship between normalized equivalent effective uniform confining stress $\left(f'_{cc}/f'_{c0}\right)$ and normalized confined peak strength $\left(f_{le}/f'_{c0}\right)$. Additionally, a design retrofitting strategy is put forth. The influence of both internal and external confinement is combined in this analytical technique, which is explored. In order to validate the retrofit methodology, an additional specimen has been constructed and examined. The outcome has demonstrated a very good performance. More extensive research on the topic is recommended to further study the use of various parameters such as grouting and also bolts and stiffeners to provide better contact and strengthen or stiffen the steel collars.

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