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AXIAL BEHAVIOR OF REINFORCED CONCRETE SHORT COLUMNS STRENGTHENED WITH WIRE ROPE AND T-SHAPED STEEL PLATE UNITS

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ABSTRACT

This paper presents a relatively simple column strengthening procedure using unbonded wire rope and T-shaped steel plate units. Twelve strengthened columns and an unstrengthened, control column were tested to failure under concentric axial load to explore the significance and shortcomings of the proposed strengthening technique. The main variables investigated were the volume ratio of wire ropes as well as geometrical size and configuration of T-shaped steel plates. Axial load capacity and ductility ratio of columns tested were compared with predictions obtained from the equation specified in ACI 318-05 and models developed for conventionally tied columns, respectively. The measured axial load capacities of all strengthened columns were higher than predictions obtained from ACI 318-05, indicating that the ratio of the measured and predicted values increased with the increase of volume ratio of wire ropes and flange width of T-shaped steel plates. In addition, at the same lateral reinforcement index, a much higher ductility ratio exhibited by strengthened columns having a volume ratio of wire ropes above 0.0039 than tied columns. The ductility ratio of strengthened columns tested increased with the increase of flange width, thickness, and web height of T-shaped steel plates. A mathematical model for the prediction of stress-strain characteristics of confined concrete using the proposed strengthening technique is developed, that was in good agreement with test results.

Keywords: strengthening, wire ropes, T-shaped steel plates, axial load capacity, ductility ratio, stress-strain relationship, confined concrete.

INTRODUCTION

Reinforced concrete columns designed and constructed before 1970s are often pointed out to be deficient in resisting lateral loads due to the lack of detailed provisions in the codes available at that time[1]. Some concrete columns in old structures have also required seismic strengthening owing to the rezoning of seismic activity of the area. Seismic performance of concrete columns could be upgraded by enhancing stiffness, strength, and ductility. In particular, a seismic rehabilitation scheme to enhance the ductility of certain critical members at the region of plastic hinges has become increasingly popular.

Bonded-type strengthening procedures using steel plates, high-strength non-metallic fiber laminates, or composite materials together with adhesives such as epoxy resin were developed and recently used in various repair and strengthening fields. Many investigations[2, 3] showed that the bonded-type strengthening procedures are effective in enhancing the strength and ductility of existing reinforced concrete columns. However, few drawbacks were also identified[4, 5] such as, debonding of external laminates from concrete surface, high cost, and poor long-term behavior of the system owing to different coefficients of thermal expansion of concrete, adhesive, and non-metallic fiber laminates. In addition, a wrapping technique using composite materials such as fiber reinforced plastic (FRP) laminates is considerably less effective for square and rectangular columns as the lateral confining pressure is developed through the membrane action of wrapping materials[1]. As a result, few investigations[1, 5] on unbonded-type strengthening method were conducted. From the axial compressive tests on columns externally strengthened with hollow structural section collars, Hussain and Driver[1] concluded that the proposed strengthening technique could enhance the axial load capacity and ductility of deteriorated reinforced concrete columns and allowed the strengthened columns to have a larger effective core area. Yang and Ashour[5] also developed a new technique using wire rope units and steel elements; they showed that the proposed strengthening method was highly economical and structurally efficient in enhancing seismic resistance of concrete columns. However, practical design and application of unbonded-type strengthening procedures would require further investigations to evaluate the influence of various parameters on the load capacity, ductility and stress-strain relationship of concrete confined by the provided strengthening procedure. The present study proposes a relatively simple strengthening procedure using unbonded wire rope and T-shaped steel plate units. Twelve strengthened columns and an unstrengthened, control column were tested to failure under concentric axial load. The volume ratio of wire ropes as well as geometrical size and configuration of T-shaped steel plates were selected as the main variables to explore the significance and shortcomings of the developed strengthening technique. A mathematical model is also proposed to evaluate the stress-strain relationship of concrete confined by wire rope and T-shaped steel plate units.

EXPERIMENTAL INVESTIGATION

Strengthening procedure

Details of unbonded-type wire rope and T-shaped steel plate units used for strengthening of reinforced concrete columns tested in the current investigation are shown in Fig. 1. This technique is a simplified version of that presented in a previous investigation[5]. A T-shaped steel plate unit, placed on each side of columns, is comprised of two corner and one or more intermediate T-shaped steel plates joined by steel bars welded at both ends of the T-shaped steel plates as shown in Fig. 1 (b). The web height of intermediate T-shaped steel plates is twice of that of corner T-shaped steel plates so that all T-shaped steel plates are fully supported by the prestressed wire rope. The function of the T-shaped steel plates is to enhance the bearing capacity of concrete against wire ropes and to prevent spalling of cover concrete. As a result, buckling resistance of internal longitudinal steel reinforcement would be enhanced and lateral confinement would be provided to both cover and core concrete of columns. A wire rope unit is composed of a wire rope, and one set of eye-bolt with washer and nut. Both ends of the wire rope are connected to a 10 mm diameter eye-bolt. Both ends of eye-bolts are passed through the holes provided at the intermediate T-shaped steel plate and then tightened by nuts as shown in Fig. 1 (b).

The prestressed force transferred to wire ropes is controlled by the torque value applied simultaneously to the nuts at both ends of the wire rope, similar to the torque control method in high-strength bolts. If torque coefficient of the eye-bolt and nuts used in the strengthening procedure is adequately evaluated, therefore, the tensile force generated in wire ropes can be reasonably calculated from[6]:

$$T = kd_b N \tag{1}$$

where d_b = bolt diameter and k = a torque coefficient dependent on the friction coefficient and geometrical conditions of the thread in bolts and nuts. To evaluate the torque coefficient of the proposed wire rope unit, forty specimens composed of eye-bolt, nut and wire rope with load cell were tested as shown in Fig. 2[7]. Based on test results, the torque coefficient k in the developed wire rope unit can be reasonably assumed as 0.3. In addition, the effect of kink of the wire rope at the corners on the tensile force exerted on wire ropes can be negligible as the torque is simultaneously applied at both end bolts. Long term losses of prestress in wire ropes due to deformation of concrete and relaxation of wire ropes would occur and should be considered when designing the prestressing tensile forces in wire rope units; however, short term losses is expected to be small and therefore neglected during the column test.

Test specimens

Twelve strengthened columns and an unstrengthened, control column, were tested to failure under concentric axial load. The geometrical details of wire rope and T-shaped steel plate units arranged in the test specimens are given in Table 1 and Fig. 3. All columns tested had stubs at both ends of 375 mm square section and 200 mm high. The stubs were strengthened with carbon fiber sheets to prevent pre-mature failure at these regions. The test zone of all columns between both stubs was 275 mm square section and 825 mm high, resulting in an aspect ratio of 3.0. The concrete core size measured from the center of the internal steel hoop was kept constant at 225 mm square. Each concrete column was longitudinally reinforced with twelve steel bars of 16 mm diameter, producing a longitudinal reinforcement ratio $p_s \left(= \dfrac{A_s}{BD}\right)$ of 0.032, where A_s = total area of the longitudinal reinforcement, and B and D = total width and depth of column section, respectively. Internal hoops of 6 mm diameter and 275 mm spacing were provided throughout the test zone of all specimens. T-shaped steel plate units at 30 mm from inner ends of both stubs were installed in the test zone as shown in Fig. 3. The corner T-shaped steel plates had the same geometrical dimensions as intermediate T-shaped steel plates except for the web height. Wire ropes used consist of six strands laid helically over a central core of a smaller independent wire rope. The nominal diameter and net area of a wire rope were 6 mm and 17.56 mm^2, respectively. In an earlier investigation, Yang and Ashour[5] concluded that the optimum prestress of wire ropes to enhance the strength and ductility of columns ranged from 0.3 to 0.4 f_{wu}, where f_{wu} = tensile strength of wire ropes. Therefore, the initial prestress applied to wire ropes of all column specimens was 534 MPa, an equivalent of 30% of wire rope tensile strength f_{wu}.

The main variables investigated were the volume ratio of wire ropes $\rho_w \left(= \dfrac{4D_w A_w}{D^2 s_w}\right)$, geometrical size and configuration of T-shaped steel plate units, where A_w = net area of a wire rope, s_w = spacing of wire ropes, and D_w = distance laterally measured from centre to centre of wire ropes as shown in Fig. 3. T-shaped steel plate units were classified into two types: L-type for two corner T-shaped steel plates and one T-shaped steel plate along every intermediate longitudinal reinforcing bar, and I-type for two corner and one intermediate T-shaped steel plates in each side of the columns tested. As a result, a total of 16 and 12 T-shaped steel plates were used for columns with L-type and I-type T-shaped steel plate units, respectively, as shown in Fig. 3.

Specimen C1 was an unstrengthened, control column. For specimen C2, the volume ratio of wire ropes was 0.0052, and the flange width, web height and thickness of intermediate T-shaped steel plates 35 mm, 40 mm, and 5 mm, respectively. Specimens C3, C4, and C5 were designed to evaluate the effect of ρ_w at the same geometrical size and configuration of T-shaped steel plates as given in Table 2. On the other hand, ρ_w of the rest of specimens was kept constant at 0.0052. Specimens C6 and C7 were identical apart from the flange width of T-shaped steel plates, whereas specimens C8 and C9 were also identical apart from the T-shaped steel plate thickness as presented in Table 2. Specimens C10 and C11 were designed to evaluate the effect of web height of T-shaped steel plates. L-type T-shaped steel plate units were installed in all strengthened column specimens but specimens C12 and C13 where I-type T-shaped steel plate units were used. Flange widths of T-shaped steel plates used in C12 and C13 were 35 mm and 45 mm, respectively.

Material properties

Fig. 4 and Table 2 show the stress-strain relationships and mechanical properties of metallic materials used in the present study, respectively. The yield strength of 6 mm diameter steel reinforcement and eye-bolt was determined from 0.2% off-set method as they showed no clear yield zone. The wire rope does not also exhibit a yield plateau as presented in Fig. 4. The elastic modulus of wire ropes used in column strengthening was nearly 65% of that of steel of 200 GPa. Based on the experimental results and analytical model using orthotropic sheet theory, Raoof and Kraincanic[8] concluded that the elastic modulus of wire ropes is significantly influenced by the interwire friction and generally falls in a range of 50~60% of that of steel.

Design compressive strength of concrete was selected to be 24 MPa, similar to old, deteriorated concrete in need of strengthening. Control specimens of 150 mm diameter × 300 mm high cylinders were cast and cured simultaneously with columns to determine the compressive strength of concrete. Concrete strength obtained from testing three cylinders for each column specimen is given in Table 1.

Instrumentation and test set-up

All columns were loaded concentrically using a 3000 kN capacity universal testing machine with a displacement rate of 0.15 mm/min. In the test region of all columns, displacement was recorded by a 50 mm capacity linear variable differential transducers (LVDT) mounted at the four corners of each column tested as shown in Fig. 5. In addition, strains in T-shaped steel plates and concrete were measured by 5 mm electrical resistance strain gages (ERS) bonded at mid-height of test zone as shown in Fig. 3. Plaster was spread out between column surface and loading plate to achieve uniform distribution of the applied load. A spherical hinge was also positioned between the testing machine head and columns to achieve concentric axial load in case of large deformation. The test was terminated when either a wire rope was fractured or the applied load suddenly dropped. All test data were captured by a data logger and automatically stored.

TEST RESULTS AND DISCUSSIONS

General behaviour

Fig. 6 presents typical crack propagation and failure mode of columns strengthened with wire rope and T-shaped steel plate units tested. Initial cracks in cover concrete commonly occurred longitudinally at 80~90% of column capacity at T-shaped steel plate location as shown in Fig. 6 (a) and Table 3. For tied columns, the separation of cover concrete from the core concrete generally occurred just before the ultimate strength of columns[9]. However, no spalling of cover concrete was detected in the strengthened columns before its ultimate strength due to the confinement effect of wire rope and T-shaped steel plate units. After the longitudinal cracks, cracks inclined at 30~45 degrees to the longitudinal axis of columns suddenly appeared at the mid-height of columns as shown in Fig. 6 (b), similar to the conical failure of concrete cylinders under concentric axial loads, and then columns reached their ultimate strength. Beyond the occurrence of inclined cracks, the load carrying capacity of columns is primarily a function of confinement provided by wire rope and T-shaped steel plate units. With the increase of axial displacement, T-shaped steel plates buckled laterally owing to the lateral expansion of confined concrete due to Poisson effect as shown in Fig. 6 (c), which eventually caused a sudden drop of the load carried by columns, and then wire ropes were ruptured as depicted in Fig. 6 (d).

Axial load versus strains

Axial strains in different columns tested against applied axial load are shown in Fig. 7. The axial strains were calculated as the ratio of the average displacement obtained from the four LVDTs at the corners of columns tested to the gage length of 825 mm. Test results of unstrengthened column C1 are also given in Fig. 7 (a). The initial stiffness of columns tested was nearly independent on the geometrical size of T-shaped steel plates, but slightly increased with the increase of volume ratio of wire ropes. The unstrenthened column C1 showed a brittle failure mode with the occurrence of inclined cracks. On the other hand, the strengthened columns having a volume ratio of wire ropes larger than 0.0039 sustained around 85~90% of their ultimate strength even after inclined cracks. The axial load capacity and lateral buckling resistance of T-shaped steel plates increased with the increase of the volume ratio of wire ropes as shown in Fig. 7 (a) and Table 3. Increasing the flange width of T-shaped steel plates caused a slight increase in the axial load capacity of columns as shown in Fig. 7 (b), as a wider flange width of T-shaped steel plates would produce a larger confined area around concrete. The thickness and web height of T-shaped steel plates had little influence on the structural behavior of columns tested until the lateral buckling of T-shaped steel plates as shown in Fig. 7 (c) and Fig. 7 (d), respectively. However, the lateral buckling resistance of T-shaped steel plates was decreased with the decrease of the flange width, thickness, and web height of T-shaped steel plates. The axial load capacity and ductility of columns tested were influenced by the configuration of T-shaped steel plates as shown in Fig. 7 (e). Axial load capacity and ductility of columns strengthened with L-type T-shaped steel plates were higher than those of columns with I-type ones at the same geometrical size of T-shaped steel plates. Razvi and Saatcioglu[10] showed that increasing the number of longitudinal reinforcement laterally supported by conventional ties could enhance the strength gain and ductility of confined concrete. Therefore, a higher lateral confinement pressure would be developed by L-type T-shaped steel plates than I-type ones. On the other hand, the lateral buckling resistance of T-shaped steel plates was nearly independent on the configuration of T-shaped steel plates.

Axial load capacity

Fig. 8 shows the average strains in T-shaped steel plates recorded for ERS gages presented in Fig. 3 and concrete against the applied axial load for test specimen C2. Strains measured for other strengthened columns were similar to those recorded for column C2, therefore not presented here. The strains in concrete increased with the increase of the axial load up to the occurrence of the inclined cracks, similar to the axial strains in columns tested shown in Fig. 7. On the other hand, T-shaped steel plates did not experience any strains with the increase of the applied axial loads until the occurrence of inclined cracks after which strains slightly increased owing to the expansion of concrete, and finally reached the yield strain at the lateral buckling of T-shaped steel plates. This indicates that T-shaped steel plates did not contribute to the transfer of axial loads as they installed shorter of the column ends and mainly play the role of concrete confinement.

Yang and Ashour[5] proposed that an axial load of columns strengthened with wire rope units and steel elements is transferred by core concrete, longitudinal reinforcement, steel elements, and cover mortar. As the T-shaped steel plates with no cover mortar can not transfer axial loads, the axial load capacity of concrete columns strengthened with the proposed technique can be predicted using the empirical equation specified in ACI 318-05[11], which combines axial load transfer capacities of concrete and longitudinal reinforcement. Comparisons of the measured and predicted axial load capacities of columns tested are given in Table 3. The axial load capacity of the unstrengthened column was 3% lower than the prediction of ACI 318-05. However, the measured axial load capacities of all strengthened columns were higher than the predictions of ACI 318-05, and the ratio of the measured and predicted values increased with the increase of the volume ratio of wire ropes and flange width of T-shaped steel plates. This may be attributed to the lateral confinement pressure provided by wire rope and T-shaped steel plate units.

Ductility ratio

The ductility of columns subjected to axial loads is generally evaluated from a ductility ratio μ defined below[9, 12]:

$$\mu = \frac{\varepsilon_{85\%}}{0.004} \tag{2}$$

where $\varepsilon_{85\%}$ = the strain value corresponding to 85% of the ultimate strength of strengthened columns, on the descending branch of the stress-strain curve of concrete. The ductility ratio obtained from Eq. (2) for columns tested is presented in Table 3. The influence of lateral reinforcement index, $\frac{\rho_w f_{ws}}{f_{co}}$ for columns strengthened on the ductility ratio of strengthened columns tested is presented in Fig. 9, where f_{ws} = notional wire rope stress at ultimate strength of columns, which is assumed as $f_{wu} - f_i$, f_i = initial tensile stress in wire ropes and f_{co} = compressive strength of unconfined concrete which is generally assumed to be $0.85 f_c'$ [10, 12]. On the same figure, the ductility ratio of tied columns confined by internal hoops, tested by Chung et al.[12] is also plotted against the lateral reinforcement index $\frac{\rho_h f_{yh}}{f_{co}}$, where ρ_h and f_{yh} = volume ratio and yield strength of internal hoop reinforcement, respectively. At the same lateral reinforcement index, a much higher ductility ratio exhibited by the strengthened columns having a volume ratio of wire ropes above 0.0039 than that in tied columns. The ductility ratio of strengthened columns increased with the increase of flange width, thickness, and web height of T-shaped steel plates as indicated in Table 3. On the other hand, the ductility ratio of strengthened columns having I-type T-shaped steel plate units was much lower than that of strengthened columns having L-type T-shaped steel plate units, as the capacity of the strengthened columns having I-type T-shaped steel plate units was sharply dropped below 85% of their ultimate strength as soon as inclined cracks formed. However, columns with I-type T-shaped steel plate units showed a highly ductile behavior after the 85% of their ultimate strength as shown in Fig. 7 (e).

STRESS-STRAIN MODEL OF CONFINED CONCRETE

Structural behavior of short concrete columns subjected to axial load or combination of axial and lateral loads is generally well characterized and predicted by the stress-strain relationship of confined concrete. Based on the empirical confinement model proposed by Razvi and Saatcioglu[10] calibrated against extensive test results on tied columns, a mathematical model for the stress-strain relationship of concrete confined by wire rope and T-shaped steel plate units is developed. Lateral confinement pressure on concrete provided by wire rope and T-shaped steel plate units is presented in Fig. 10. The equivalent uniform confinement pressure f_{le} provided by wire rope and T-shaped steel plate units can be assumed as follows[10]:

$$f_{le} = k_1 \rho_w f_{wcc} \tag{3}$$

where k_1 = a coefficient for determining the efficiency of spacing of wire ropes as well as flange width and configuration of T-shaped steel plates, and f_{wcc} = tensile stress in wire ropes at the peak stress of confined concrete. The coefficient k_1 proposed by Razvi and Saatcioglu[10] using regression analysis is extended below to account for concrete confined by wire rope and T-shaped steel plate units:

$$k_1 = 0.15\sqrt{\left(\frac{D}{s_w}\right)\left(\frac{D}{C}\right)} \leq 1.0 \tag{4}$$

where C = lateral distance between centroids of overhanging flanges of T-shaped steel plates as shown in Fig. 10.

It is so difficult to evaluate the tensile stress f_{wcc} in wire ropes at the peak stress of confined concrete at this stage. Many researchers[10, 12, 13] pointed out that most of high-strength hoop reinforcement did not yield at the peak stress of concrete in tied columns. Sakino and Sun[13] limited the stress of hoop reinforcement at the peak stress of concrete below 700 MPa. Razvi and Saatcioglu[10] showed that the stress development in high-strength hoop reinforcement mainly depends on the volume ratio and efficiency of hoop reinforcement, as well as concrete strength, and also derived the tensile stress in hoop reinforcement from regression analysis using a large volume of test data. Hence, the use of the tensile strength of wire ropes would overestimate the peak stress of confined concrete, as the wire ropes had a high tensile strength above 1790 MPa as given in Table 2. Eq. (5) below is developed as a modification to the empirical model of Razvi and Saatcioglu[9] to evaluate the tensile stress f_{wcc} in wire ropes at the peak stress of concrete:

$$f_{wcc} = E_w\left(0.0025 + 0.04\sqrt[3]{\frac{k_1 \rho_w}{f_{co}}}\right) \leq f_{ws} \tag{5}$$

where E_w = elastic modulus of wire ropes.

The stress-strain curve for confined concrete, represented in Fig. 11, can be generally expressed by Eq. (6) below using the Popovics' s model[14] for concrete.

$$f_c = \frac{f_{cc}\left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right) r}{(r-1) + \left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right)^r} \tag{6}$$

where f_c and ε_c = stress and strain of confined concrete, respectively, f_{cc} and ε_{cc} = peak stress and the corresponding strain of confined concrete, respectively, and r = a coefficient related to the gradient of the descending branch of the stress-strain curve. The peak stress f_{cc} of confined concrete is generally expressed as $K_s f_{co}$ [10, 13, 14], where K_s = strength gain factor of confined concrete, which is a function of concrete strength as well as effective lateral pressure provided by lateral reinforcement. Therefore, the peak stress and the corresponding strain of concrete confined by wire ropes and T-shaped steel plate units can be derived from regression analysis of test results of Yang and Ashour[5], and this study as follows:

$$f_{cc} = K_s f_{co} = \left(1 + 1.2\frac{f_{le}}{f_{co}}\right) f_{co} \tag{7}$$

$$\varepsilon_{cc} = \left(1 + 1.48\frac{f_{le}}{f_{co}}\right)\varepsilon_o \tag{8}$$

where ε_o = strain at peak stress of unconfined concrete. Although, experimental values for ε_o can be available, empirical equation proposed by Chung et al.[5] is used in the present study as follows:

$$\varepsilon_o = \left(2100 + 11.6 f_c^{'}\right) \times 10^{-6} \tag{9}$$

To consider the influence of effective confinement pressure on the ductility of concrete, the coefficient r in Eq. (6) can be expressed as below:

$$r = \left[\frac{(E_{\sec})_{cc}}{(E_{\sec})_{cc} - (E_{\sec})_{85}}\right]^{k_2} \tag{10}$$

$$k_2 = [K_s]^{-(E_c/(E_{sec})_{cc})} \tag{11}$$

where $(E_{sec})_{cc} = \dfrac{f_{cc}}{\varepsilon_{cc}}$ and $(E_{sec})_{85} = \dfrac{0.85 f_{cc}}{(\varepsilon_{cc})_{85}}$ are the secant modules of elasticity of confined concrete as shown in Fig. 11, $(\varepsilon_{cc})_{85}$ = strain corresponding to 85% of peak stress of confined concrete on the descending branch, and E_c = initial elastic modulus of unconfined concrete, which can be obtained from $4700\sqrt{f_{co}}$ recommended by ACI 318-05, where f_{co} is in (MPa). The $(\varepsilon_{cc})_{85}$ of concrete confined by wire rope and T-shaped steel plate units can also be derived from the regression analysis of test results carried out by Yang and Ashour[5], and this study as below:

$$(\varepsilon_{cc})_{85} = \left[3.6\left(\frac{f_{le}}{f_{co}}\right)^{0.24}\right]\varepsilon_{cc} \tag{12}$$

COMPARISONS OF PREDICTIONS AND TEST RESULTS

The load carried by concrete confined by wire rope and T-shaped steel plate units at each axial strain increment is obtained from the difference between the total applied load and load resisted by longitudinal reinforcement, and then this load is converted to a stress, similar to the case of tied columns[10, 13, 14]. Comparisons of predicted and measured peak stresses and the corresponding strains of confined concrete are given in Table 4. On the same table, predictions obtained from the empirical equations proposed by Chung et al.[13], Razvi and Saatcioglu[10], and Sakino and Sun[14] based on test results of tied columns confined by internal hoops are also presented. The predictions obtained from Sakino and Sun's equation overestimate the test results owing to the excessively high tensile stress of wire ropes assumed at the peak stresses of concrete. The peak stresses of confined concrete predicted by Chung et al.'s, and Razvi and Saatcioglu's equations are slightly lower than the experimental values by an average of 5~10%. The strains corresponding to the peak stresses of confined concrete are slightly overestimated by Chung et al.'s equation and underestimated by

Razvi and Saatcioglu's equation. On the other hand, the predictions obtained from the equations proposed in the present study show a slightly better agreement with test results, indicating that the average and standard deviation of the ratio between experimental and analytical values are 0.981 and 0.026, respectively, for the peak stresses, and 0.999 and 0.041, respectively, for the strains corresponding to the peak stresses.

Fig. 12 shows comparisons between measured and predicted stress-strain curves of concrete confined by wire rope and T-shaped steel plate units. The axial load-strain relationship of columns tested is little influenced by the thickness and web height of T-shaped steel plates as shown in Fig. 7 (c) and (d), therefore the stress-strain curves of columns C8, C9, C10 and C11 are not presented in Fig. 12. In addition, the stress-strain curves predicted by Sakino and Sun's equation are not presented on the same figure as they highly overestimate experimental results as given in Table 4. Chung et al.'s, and Razvi and Saatcioglu's models highly underestimate the descending branch after the peak stress of the confined concrete, and the difference increases with the increase of volume ratio of wire rope and flange width of T-shaped steel plates. In addition, Chung et al.'s model underestimates the ascending branch up to the peak stress of the confined concrete. On the other hand, the stress-strain curves predicted from the model proposed in the current investigation are in good agreement with test results, indicating that the column ductility is highly enhanced by lateral confinement pressure provided by wire rope and T-shaped steel plate units. However, further experimental results to validate the proposed stress-stain curve would be required as lateral confinement pressure provided by wire rope and T-shaped steel plate units is influenced by various parameters such as concrete strength, amount, spacing and prestress of wire ropes, and shape and distribution of T-shaped steel plate.

CONCLUSIONS

Twelve reinforced concrete columns strengthened using wire rope and T-shaped steel plate units were tested under concentric axial loads. Based on the test results, a mathematical model for the stress-strain relationship of concrete confined by the strengthening technique was proposed. The following conclusions may be drawn:

1. The initial stiffness of columns tested was nearly independent of the geometrical size of T-shaped steel plates, but slightly increased with the increase of volume ratio of wire ropes.
2. The axial load capacity of strengthened columns slightly increased with the increase of volume ratio of wire ropes, while it was nearly independent on the thickness and web height of T-shaped steel plates.
3. The measured axial load capacities of all strengthened columns were slightly higher than predictions obtained from ACI 318-05, indicating that the ratio of the measured and predicted values increased with the increase of volume ratio of wire ropes and flange width of T-shaped steel plates.
4. Axial load capacity and ductility of columns strengthened with L-type T-shaped steel plates were higher than those of columns with I-type ones at the same geometrical size of T-shaped steel plates.
5. At the same lateral reinforcement index, a much higher ductility ratio exhibited by strengthened columns having a volume ratio of wire ropes above 0.0039 than tied columns. The ductility ratio also increased with the increase of flange width, thickness, and web height of T-shaped steel plates.
6. The stress-strain curves predicted by the proposed model were in good agreement with test results, agreeing with that the column ductility is highly enhanced by lateral confinement pressure provided by wire rope and T-shaped steel plate units. <u>However, additional test</u>

results on the proposed strengthening system would be required to validate the proposed stress-stain model.

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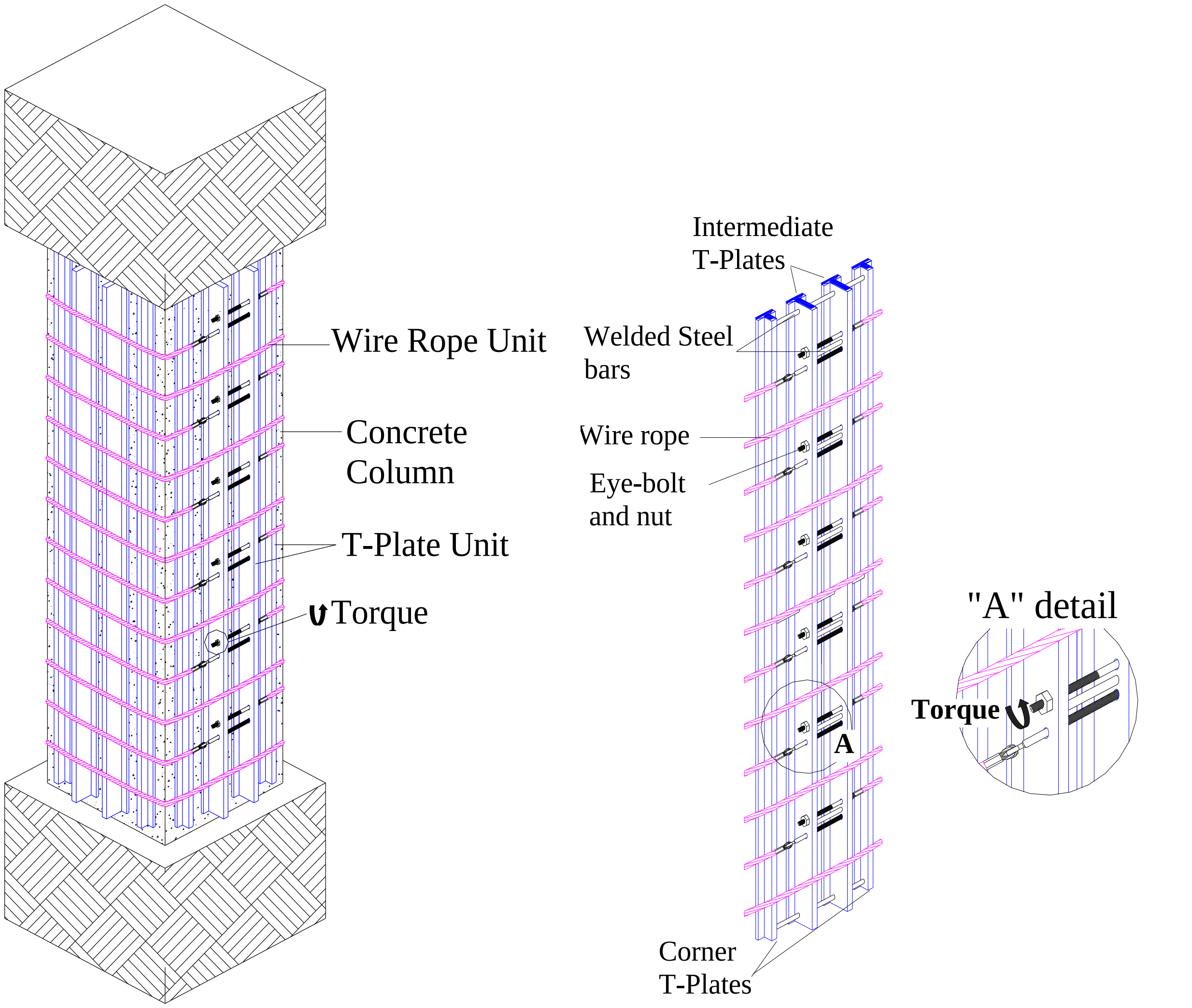
List of Figures:



(a) Strengthened columns (b) Wire rope and T-shaped steel plate units

Fig. 1- Details of proposed column strengthening procedure.

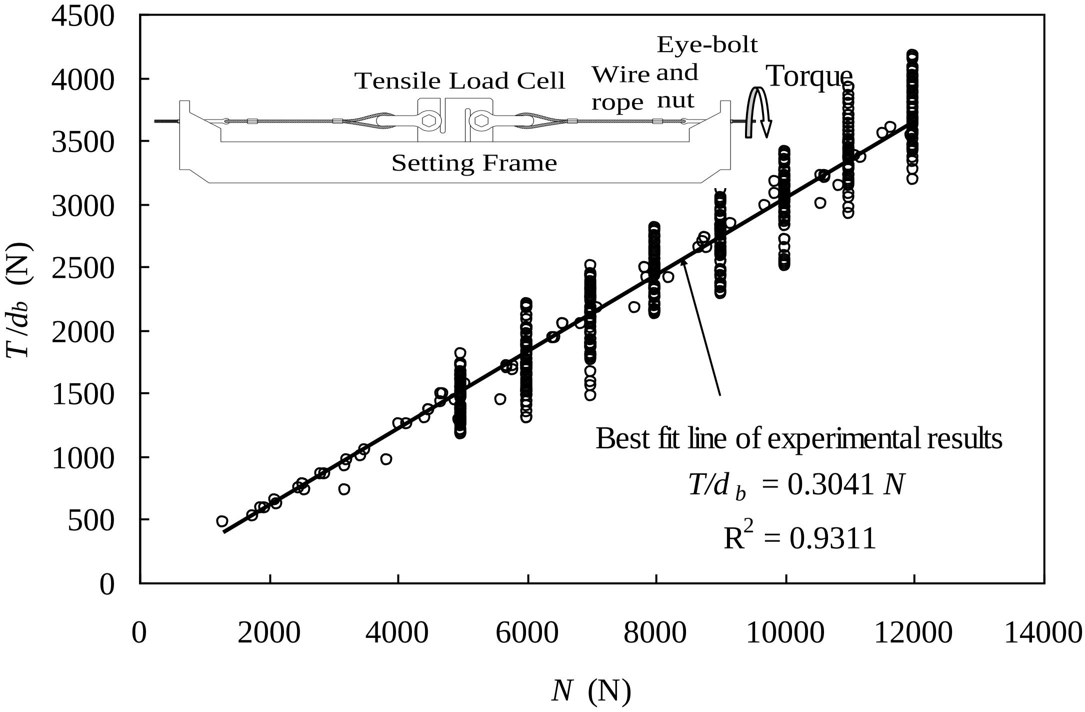
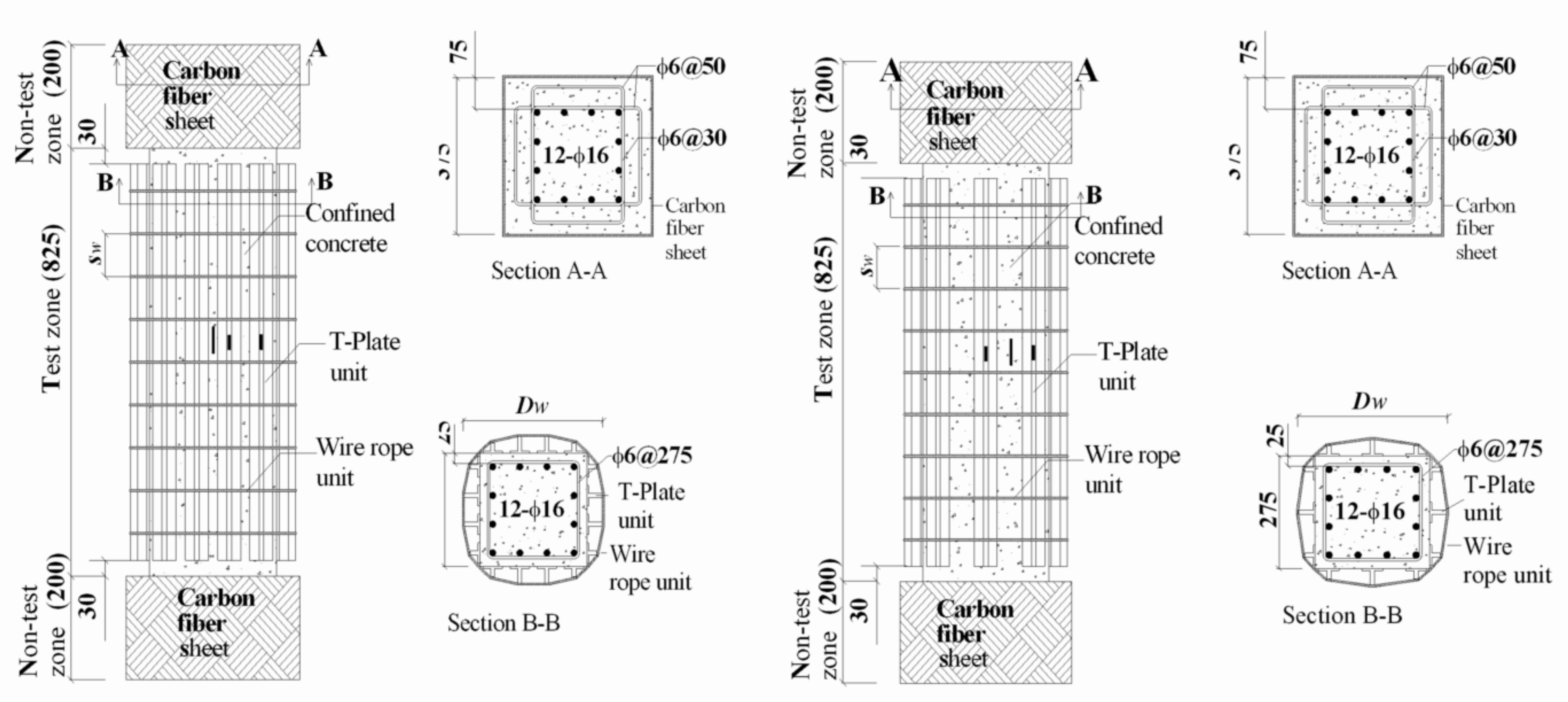


Fig. 2-Relation of N and T / d_b in a wire rope unit.



(a) Columns with L-type T-shaped steel plate

(b) Columns with I-type T-shaped steel plate

Fig. 3-Specimen details and arrangement of wire rope and T-shaped steel plate units.

(all dimensions are in mm)

(■ indicates positions of ERS gages bonded to T-shaped steel plates)

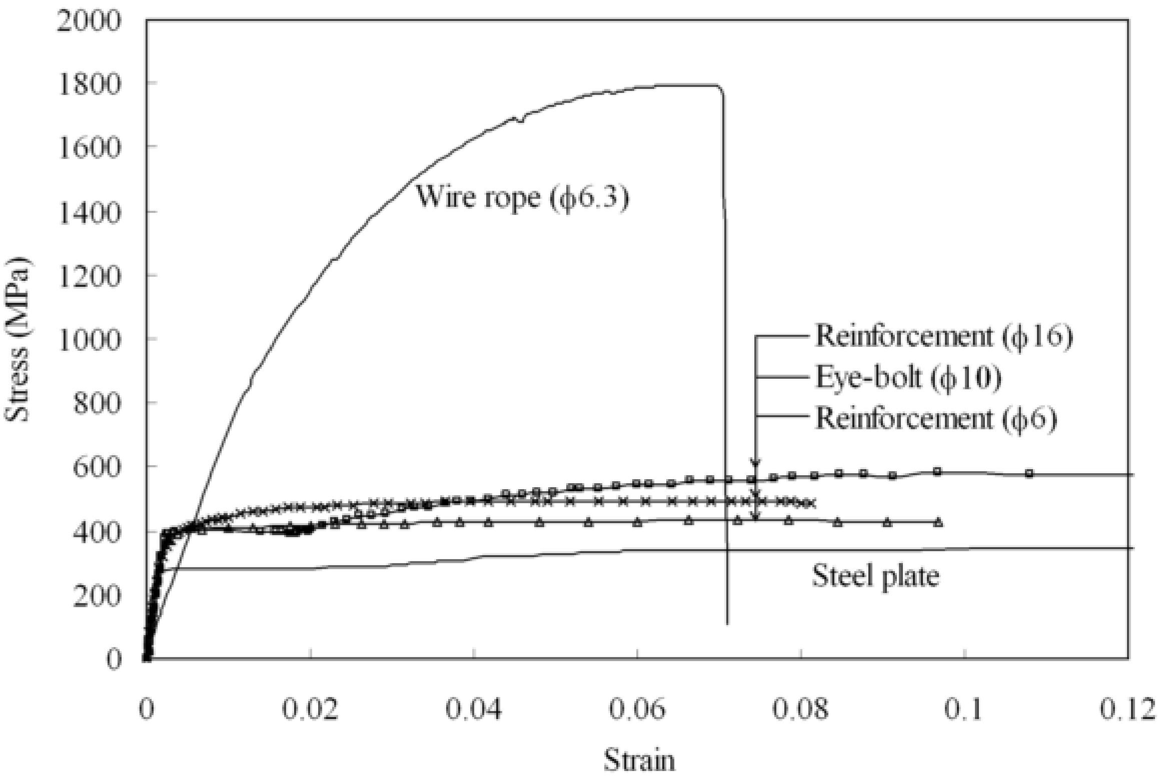


Fig. 4-Stress-strain relationships of metallic materials.

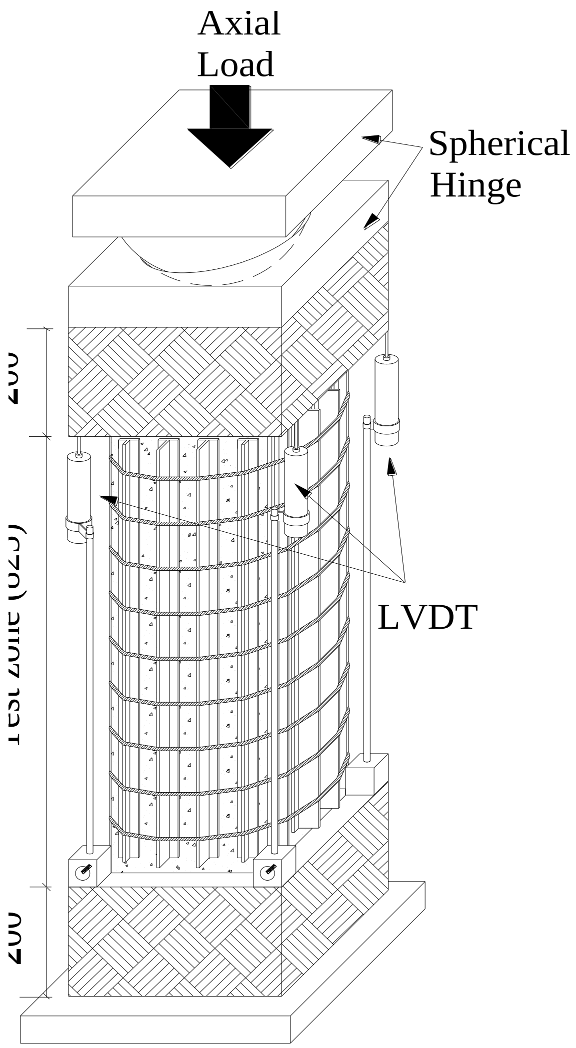
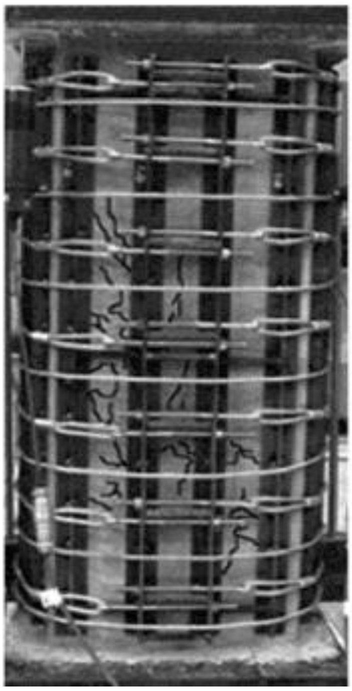


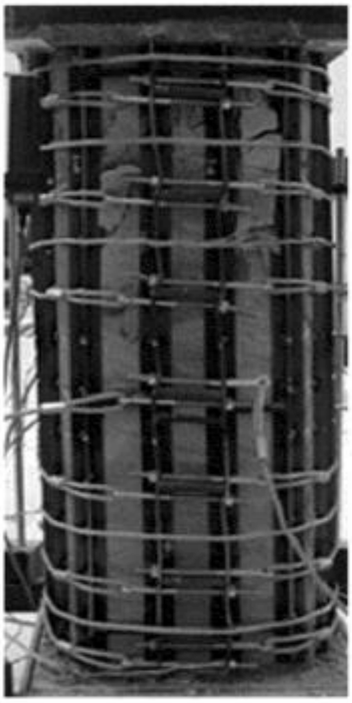
Fig. 5- Test set-up and instrumentation for test region (All dimensions are in mm).

(a) Crack propagation at cover concrete



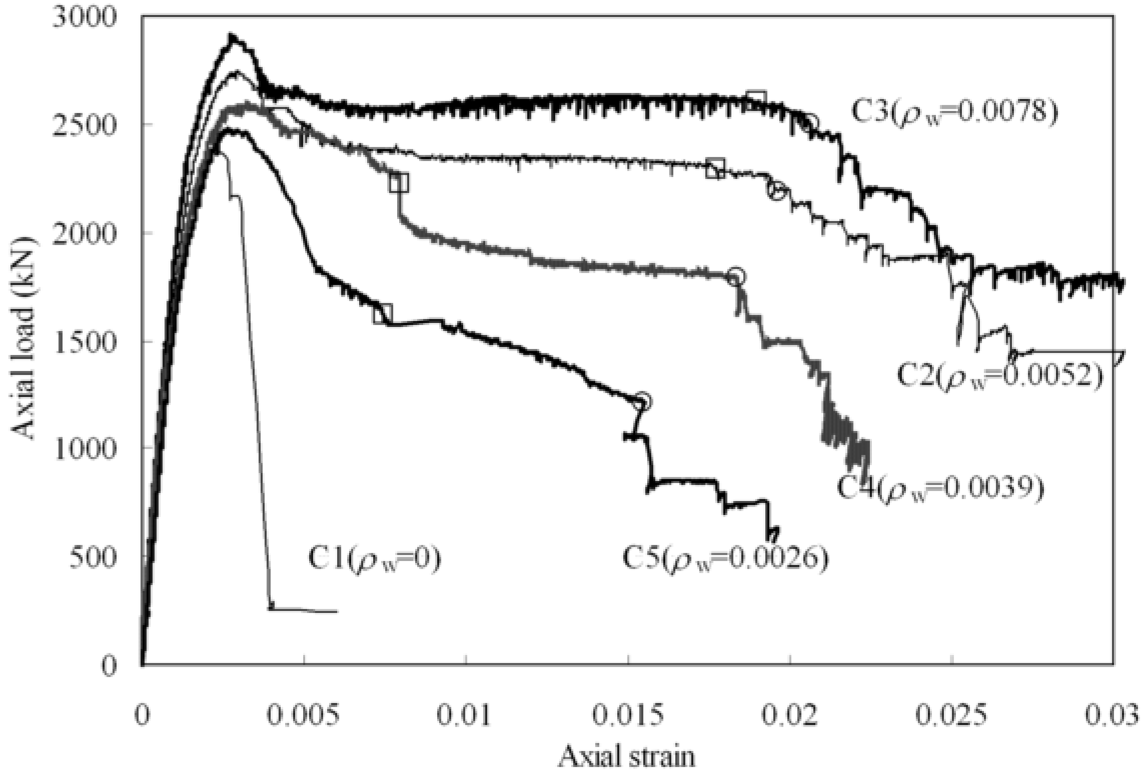
(b) Inclined cracks at ultimate strength

(c) Lateral buckling of T-shaped steel plates

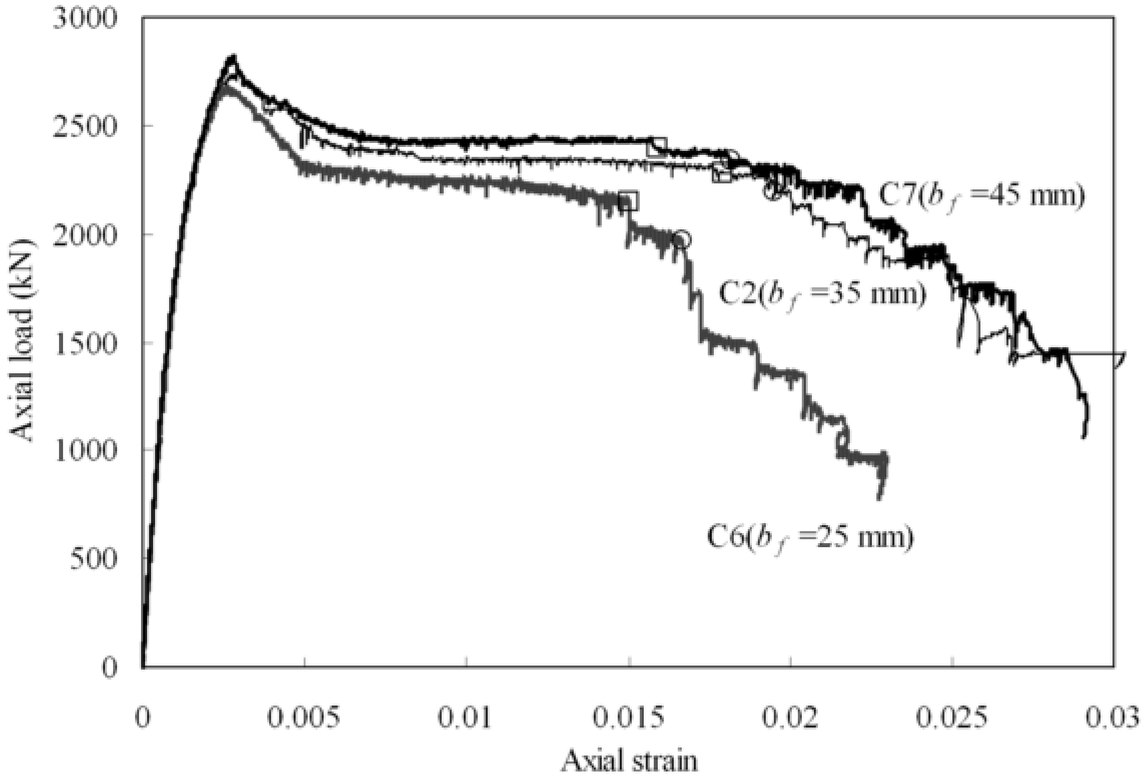


(d) Rupture of wire rope

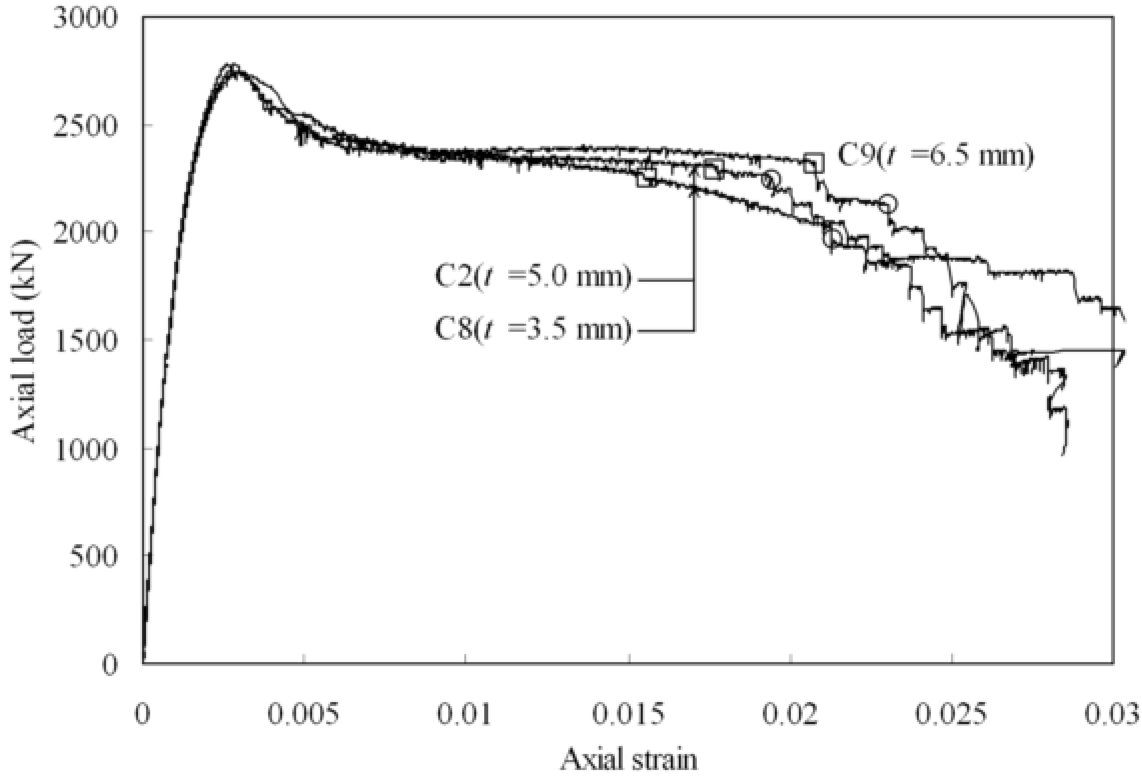
Fig. 6- Typical behaviour of strengthened columns tested (Column specimen C2).



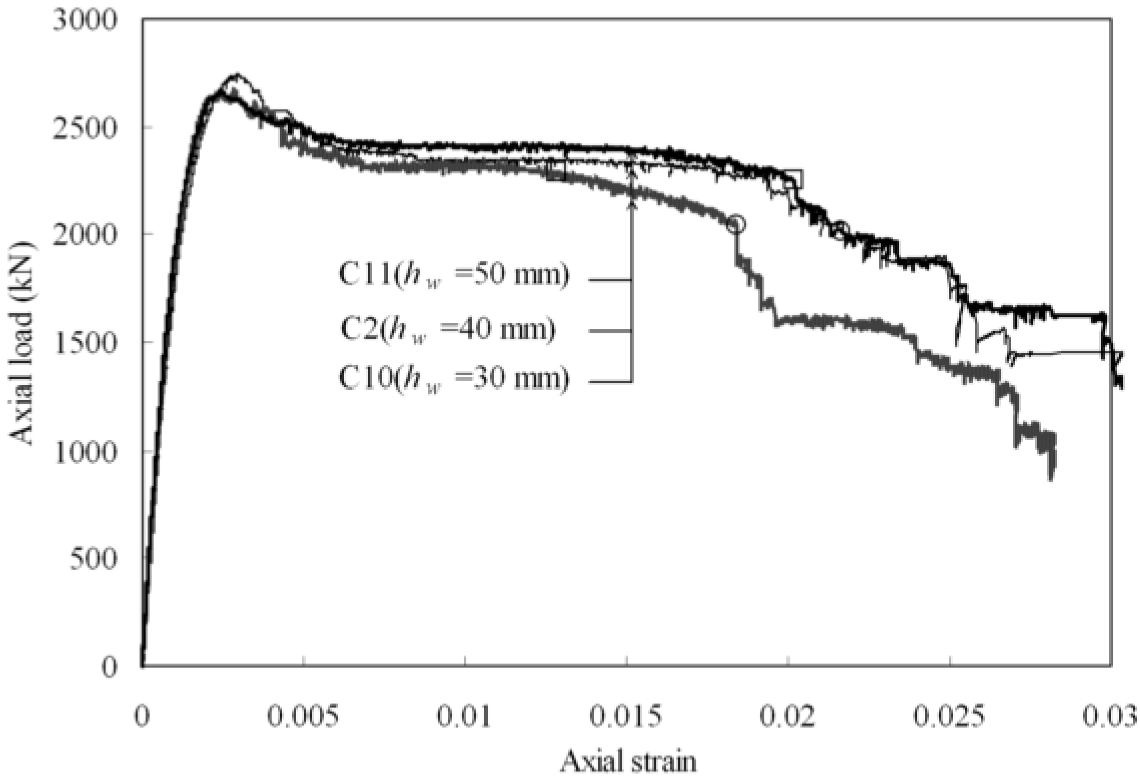
(a) Volume ratio of wire ropes



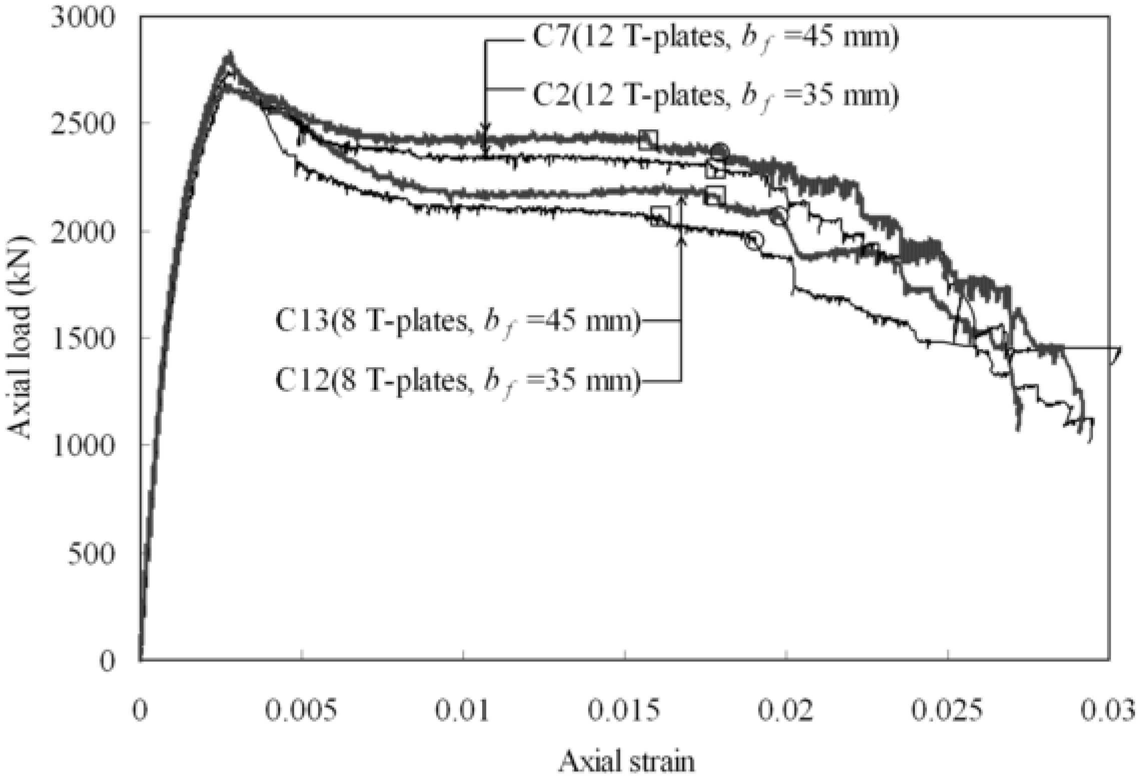
(b) Flange width of T-shaped steel plates



(c) Thickness of T-shaped steel plates



(d) Web height of T-shaped steel plates



(e) Configuration of T-shaped steel plates

Fig. 7-Axial strain against applied axial load.

(Symbols □ and ○ indicate the onset of the lateral buckling of T-shaped steel plates and rupture of wire ropes, respectively)

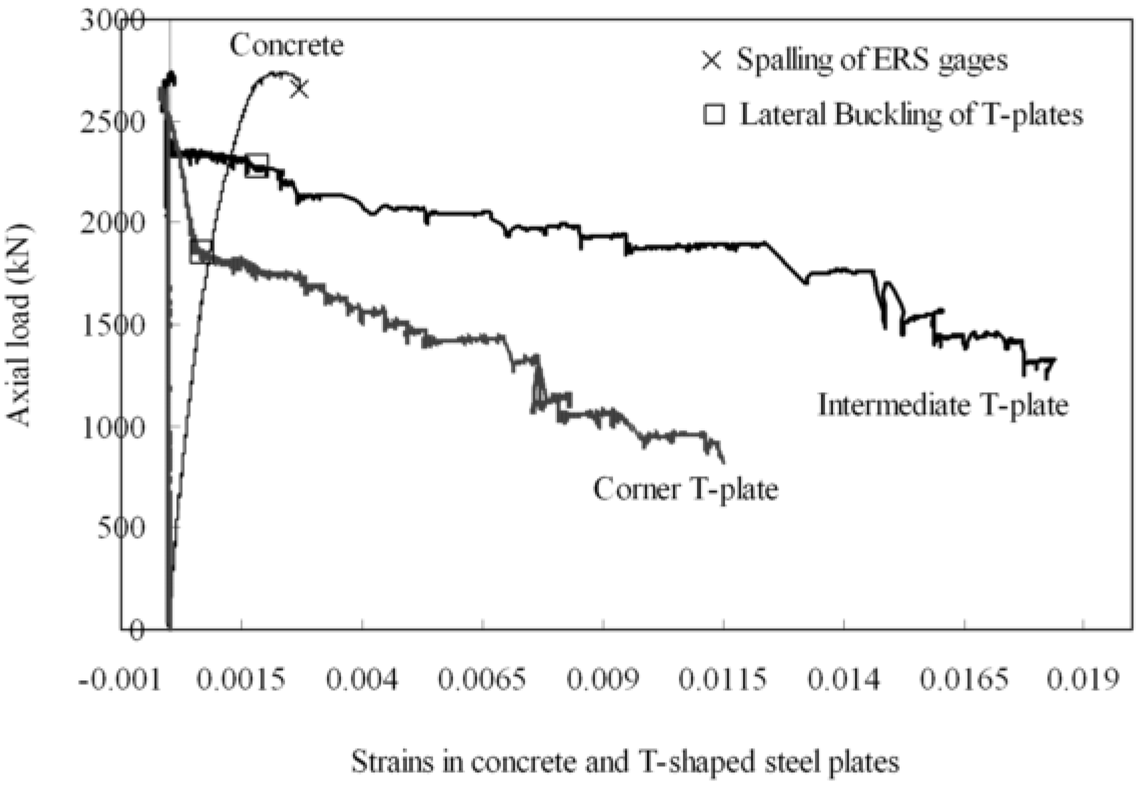


Fig. 8 -Axial strains in concrete and T-shaped steel plates against axial load for column C2.

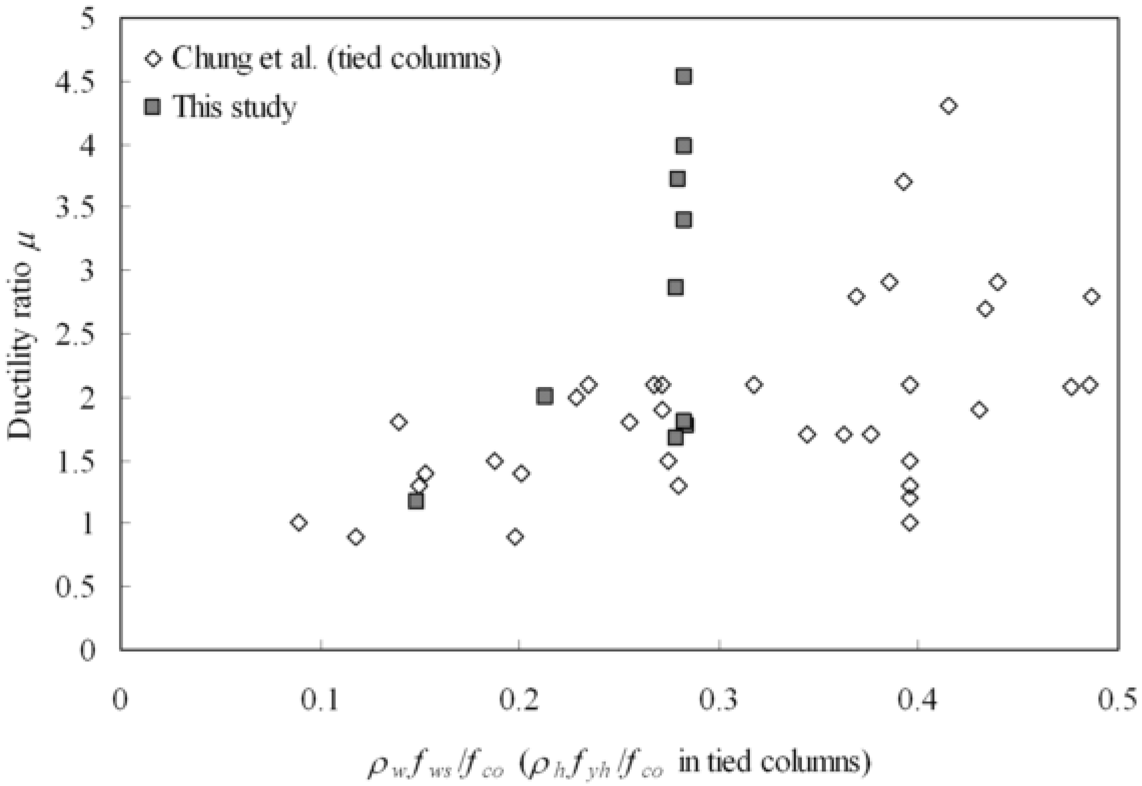
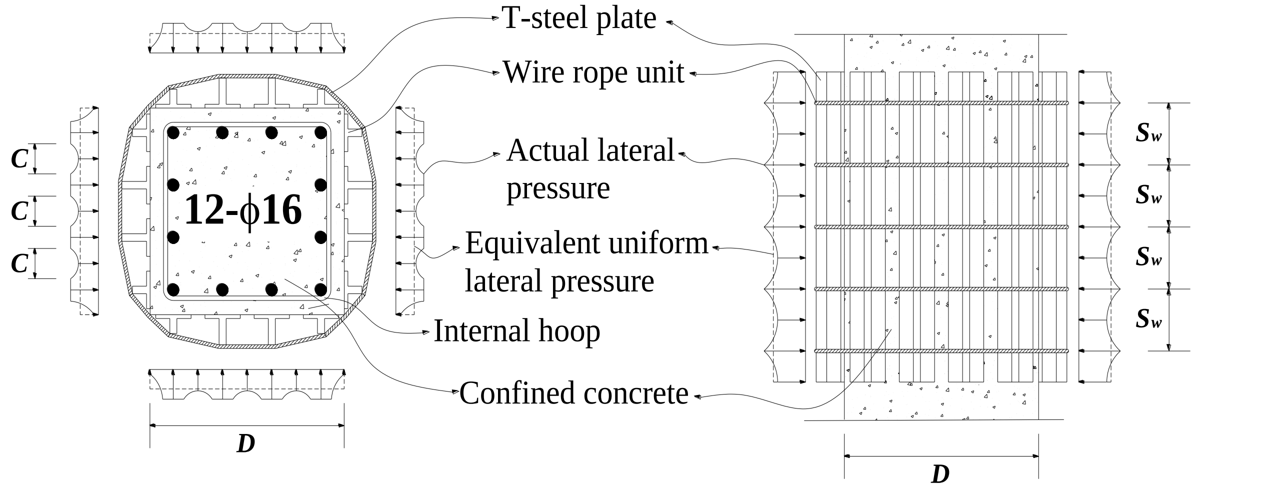


Fig. 9-Effect of lateral reinforcement index on ductility ratio of columns.



(a) Cross-section of strengthened column (b) Longitudinal view of strengthened column

Fig. 10-Idealized equivalent uniform confinement pressure.

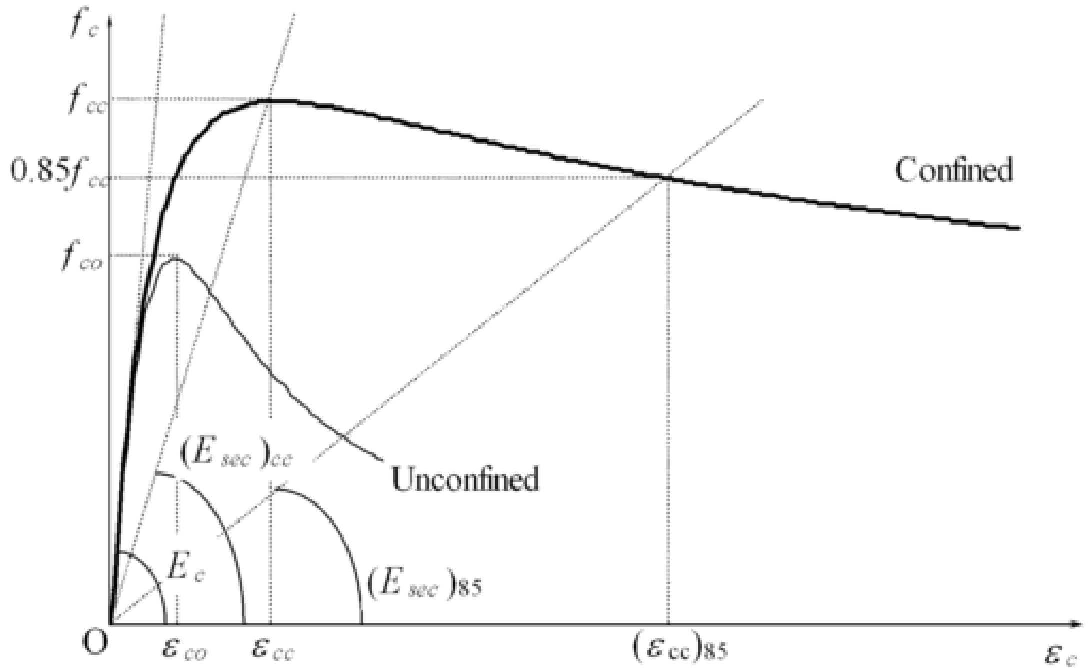
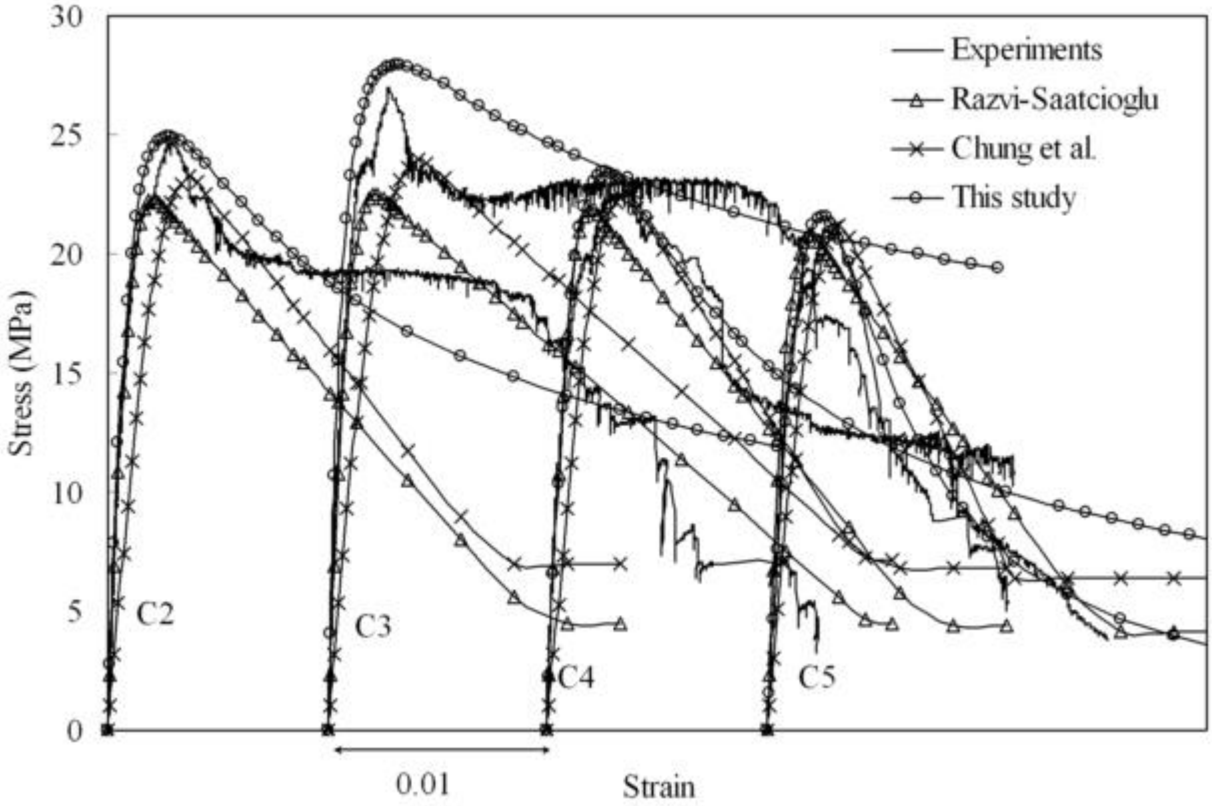
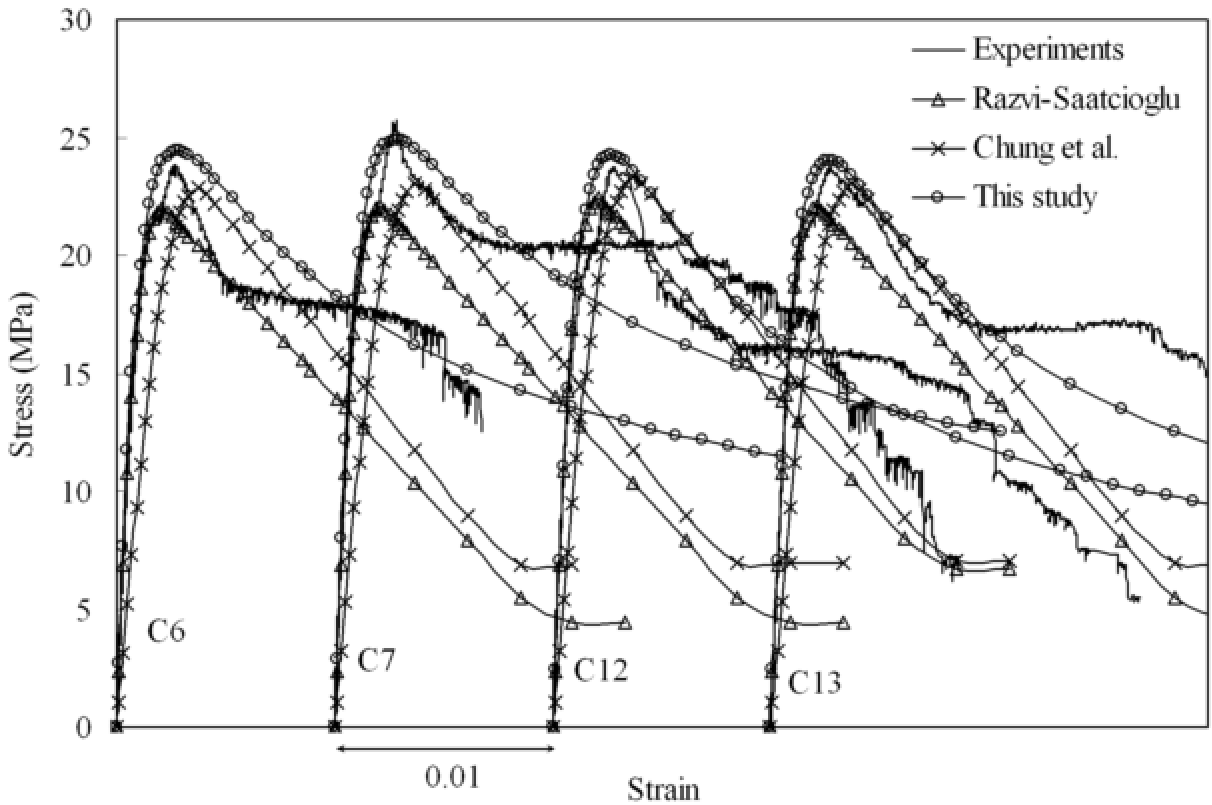


Fig. 11–Qualitative stress-strain relationships of confined and unconfined concrete.



(a) Volume ratio of wire ropes



(b) Flange width and configuration of T-shaped steel plates

Fig. 12–Comparisons of analytical and experimental stress-strain curves.