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Flexural Behavior of Reinforced Concrete Columns Strengthened with Wire Rope and T-Plate Units

by Jae-Il Sim and Keun-Hyeok Yang

This study presents the flexural behavior of reinforced concrete columns strengthened with unbonded wire rope and T-shaped steel plate units. Seven strengthened columns and an unstrengthened column were tested to failure under constant axial load and cyclic lateral loads to explore the significance and limitations of the strengthening procedure developed for resistance against earthquakes. The main variables investigated were the volume ratio of wire rope, axial load level, and the presence of mortar cover for strengthening steel elements. In addition, the theoretical monotonic lateral load-displacement curve for strengthened columns is simply derived using the combination of section laminae method and the idealized curvature-displacement relationship. The flexural capacity of columns strengthened without mortar cover was slightly higher than that of the unstrengthened column. On the other hand, the flexural capacity of strengthened columns with a 60 mm (2.36 in.) thick mortar cover was at least 2.5 times higher than that of the comparable strengthened columns without mortar cover. The developed strengthening procedure was particularly effective in enhancing the ductility of the columns, showing that the displacement ductility ratios and work damage indicators in the strengthened columns were much higher than in the unstrengthened column. The monotonic lateral load-displacement relationship of the column specimens predicted from the proposed numerical analysis is in good agreement with backbone curves obtained from measured cyclic lateral load-displacement relationships. ACI 318-05 underestimates the flexural capacity of the strengthened columns, however, as the confinement effect is not considered in the equivalent stress block of concrete specified in ACI 318-05.

Keywords: columns; confinement; ductility; flexural capacity; strengthening; wire rope.

INTRODUCTION

Reinforced concrete columns carrying axial compressive loads with or without moment require enough ductility to withstand large deformations and resist applied loads. It is also generally recognized[1,2] that the design concept of "strong column/weak beam" should be adopted for most framed structures to endure earthquakes. Hence, the seismic performance of concrete structures can be upgraded by enhancing the stiffness, strength, and ductility of columns. Reinforced concrete columns constructed before the 1970s, however, are often considered deficient in resisting lateral loads because of the lack of detailed relevant provisions in the codes available at that time.[3] Some concrete columns in old structures have also required seismic strengthening owing to the rezoning of seismic activity of the area. The seismic rehabilitation scheme aiming to enhance the ductility of concrete columns has become one of the most serious issues because it has been observed that the entire collapse of some concrete structures was caused by the failure of columns by chain action.

External strengthening for reinforced concrete columns is commonly classified into two categories: bonded type and unbonded type. Many column-strengthening techniques[4-6] using steel plates; high-strength nonmetallic fiber laminates; or composite materials, together with adhesives such as epoxy resin, have been developed and have recently been applied to various repair and strengthening fields. Several drawbacks, however, have also been identified in the bonded-type strengthening technique,[7,8] such as debonding of external laminates from a concrete surface, dust pollution from grinding of concrete surfaces, and poor long-term behavior of the system caused by different coefficients of thermal expansion of concrete, adhesive, and nonmetallic fiber laminates. In addition, Hussain and Driver[3] pointed out that a wrapping method using composite materials such as fiber-reinforced polymer (FRP) laminates is considerably less effective for square and rectangular columns, as a lateral confining pressure is developed through the membrane action of the wrapping materials without flexural stiffness. As a result, unbonded-type strengthening procedures[3,7,9] have been developed recently and have become increasingly attractive. Hussain and Driver[3] tested concrete columns externally strengthened with hollow structural section collars and concluded that the proposed strengthening technique allowed the strengthened columns to have a larger confined core area, which enhanced the axial load capacity and ductility of the reinforced concrete column specimens. Yang and Ashour[7] and Yang et al.[9] also developed a new strengthening procedure using wire rope and steel plate units and proposed a mathematical model to evaluate the stress-strain relationship of concrete confined by this strengthening technique, based on the test results of strengthened columns subjected to concentric axial load.

The objective of this study is to evaluate the flexural performance of reinforced concrete columns strengthened with unbonded wire rope and T-shaped steel plate units. Seven strengthened columns and an unstrengthened column were tested to failure under constant axial load and cyclic lateral loads. A simplified theoretical monotonic lateral load-displacement curve for strengthened columns is also derived using a combination of the section laminae method and the idealized curvature-displacement relationship[10] for comparisons with backbone curves of the measured cyclic lateral load-displacement relationship. The flexural capacity of columns tested is compared with predictions using stress blocks specified in ACI 318-05[11] and obtained from the proposed numerical lateral load-displacement relationship.

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Jae-Il Sim *is a Doctoral Student at Mokpo National University, Mokpo, Jeonnam, South Korea. His research interests include flexure, ductility, and strengthening of reinforced concrete structures.*

Keun-Hyeok Yang *is an Assistant Professor at Mokpo National University. He received his MSc and PhD from Chung-Ang University, South Korea. His research interests include ductility, recycling, strengthening, plasticity, and shear of reinforced eco-friendly concrete structures.*

RESEARCH SIGNIFICANCE

Although unbonded techniques to strengthen reinforced concrete columns have become increasingly attractive, very few experimental investigations on flexural behavior of such strengthened columns are available in the literature. Test results and numerical analysis presented in this study confirm that the strengthening procedures developed using unbonded wire rope and T-shaped steel plate units are very effective in enhancing the flexural performance of existing reinforced concrete columns.

Table 1—Details of test specimens

	Column		Mortar cover		Wire rope		Axial load	
Specimen	f'_c, MPa	A_g, mm^2	f_{cm}, MPa	A_{gm}, mm	s_w, mm	ρ_w, %	P, kN	P/P_0
C0.4-0	26.4	52,900	None	None	None	None	558	0.4
C0.4-40	25.7				40	0.97	544	0.4
C0.4-60	23.0				60	0.64	486	0.4
C0.4-80	26.2				80	0.48	555	0.4
C0.25-60	25.1				60	0.64	332	0.25
C0.55-60	25.9				60	0.64	755	0.55
M0.4-60	27.9		23.4	69,600	60	0.64	1245	0.4
M0.4-80	27.7				80	0.48	1241	0.4

Note: 1 MPa = 145 psi; 1 kN = 0.2248 kips; 1 mm = 0.0394 in.

EXPERIMENTAL PROGRAM

Test specimen details

Seven strengthened columns and an unstrengthened column were tested to failure. Full details on the strengthening procedure developed by Yang et al.[9] using unbonded wire rope and T-shaped steel plate units are presented in a companion paper. The details of wire rope and T-shaped steel plate used in the test specimens are given in Table 1 and Fig. 1. The geometrical dimensions of the column sections, the arrangement of longitudinal reinforcement, and the inner hoop bar are also shown in Fig. 1. Before strengthening, all columns had a 230 mm (9.05 in.) square section and were 1060 mm (41.73 in.) high. They were cast integrally with a 450 x 450 x 200 mm (17.71 x 17.71 x 7.87 in.) top stub and a 450 x 1250 x 500 mm (17.71 x 49.21 x 19.68 in.) bottom stub representing the column base. The column and top stub regions of the specimen represent the part of a column between the section of maximum moment and the point of contraflexure in a regular building frame. Each concrete column was longitudinally reinforced with 12 deformed bars of 13 mm (0.51 in.) diameter, producing a longitudinal reinforcement ratio p_s (= A_s/BD) of 0.029, where A_s is the total area of the longitudinal reinforcement, and B and D are the width and depth of column section, respectively. Round bars of 8 mm (0.31 in.) diameter were provided as internal hoops at spacing of 230 mm (9.05 in.) throughout the column zone of all specimens. T-shaped steel plate units isolated at 20 mm (0.79 in.) from the inner ends of both stubs were installed with the prestressed wire rope in the column zone,

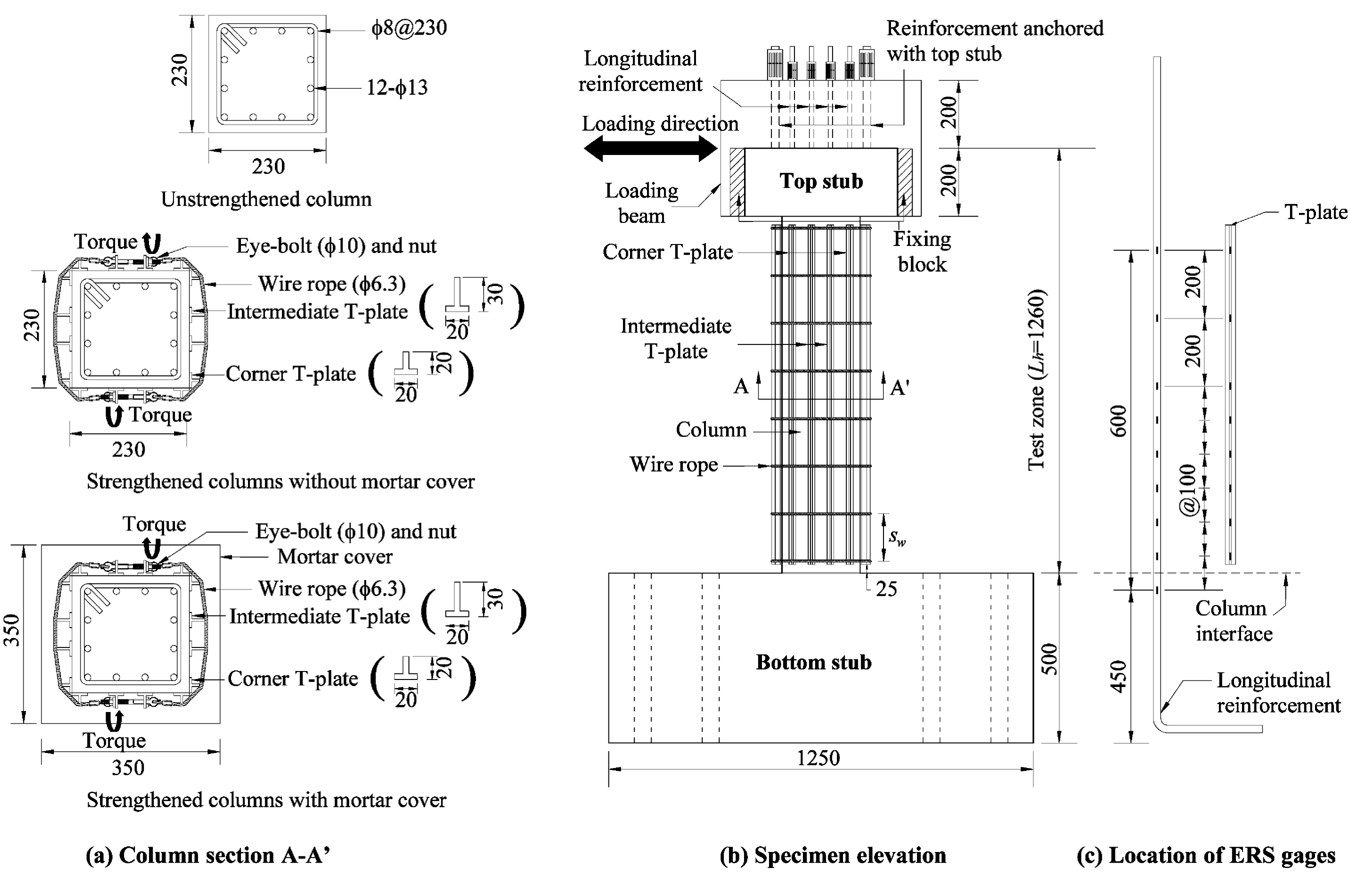

Fig. 1—Specimen details and arrangement of wire rope and T-plate units. (Note: all dimensions in mm; 1 mm = 0.0394 in.)

Table 2—Mechanical properties of metallic materials

Type	Diameter, mm	A_{net}, mm^2	f_y, MPa	ε_y	f_u, MPa	ε_u	E_s, MPa
Reinforcement	13	127	425	0.0022	594	0.267	193.2
	8	50.24	518	0.0047	572	0.276	194.7
Steel plate	—	—	284	0.0014	381	0.304	202.8
Eye-bolt	10	78.5	433	0.00409	520	0.241	207.2
Wire rope	6.3	18.6	—	—	1702	0.044	125.2

Note : A_{net} = net area, f_y = yield strength, ε_y = yield strain, f_u = tensile strength, ε_u = ultimate strain at tensile strength, and E_s = elastic modulus. 1 MPa = 145 psi; 1 mm = 0.0394 in.

as shown in Fig. 1. The corner T-shaped steel plates had the same geometrical dimensions as intermediate T-shaped steel plates of 20 x 30 x 5 mm (0.79 x 1.18 x 0.19 in.), except for two-thirds web height. A wire rope unit was composed of a wire rope and one set of eye-bolts with washer and nut. Both ends of the wire rope were connected to a 10 mm (0.39 in.) diameter eye-bolt. The nominal diameter and net area of the wire rope were 6.3 mm (0.25 in.) and 18.6 mm^2 (0.03 in.^{2}), respectively. In all strengthened columns, the initial prestress transferred to wire ropes in all strengthened columns, which can be controlled by the torque value applied simultaneously to the nuts at both ends of the wire rope, was 680 MPa (98.6 ksi), equivalent to 40% of the tensile strength of the wire rope.

The spacing of wire rope, axial load level, and presence of mortar cover for strengthening steel elements were selected as the main variables as given in Table 1. The spacing of the wire rope in the strengthened columns ranged from 40 to 80 mm (1.57 to 3.15 in.), which results in volume ratios of wire ropes ρ_w (= $4D_wA_w/BDs_w$) between 0.0097 and 0.0048, where A_s is the net area of a wire rope, s_w is the wire rope spacing, and D_w is the lateral center-to-center distance of the wire ropes. The ratio of axial load P applied to the column and axial load capacity ($P_0 = f_c'A_g + f_{cm}A_{gm}$) of the concrete column and mortar cover neglecting the axial load transfer capacity of the longitudinal reinforcement varied from 0.25 to 0.55, where f_c' and f_{cm} are compressive strength (in MPa) of concrete and mortar, respectively, and A_g (= BD) and A_{gm} are gross area of column section and area (in mm^2) of the mortar cover, respectively. The axial load capacity P_0 of columns without mortar cover is $f_c'A_g$. The specimens designed to investigate the effect of cover on the flexural behavior of strengthened columns were covered with 60 mm (2.36 in.) thick mortar, as shown in Fig. 1(a).

The specimen notation in Table 1 includes three identifiers for the selected parameters. The first part is used to identify the presence of mortar cover for the strengthening steel elements: C and M for columns with and without mortar cover, respectively. The second and third identifiers refer to the axial load level and spacing of wire rope, respectively. For example, Specimen C0.4-0 indicates an unstrengthened column having an axial load of $0.4P_0$, and Specimen M0.4-40 indicates a mortar-covered strengthened column having an axial load of $0.4P_0$ and wire rope unit spaced at 40 mm (1.6 in.).

Material properties

The concrete compressive strength of test specimens was designed to be as low as 24 MPa (3.48 ksi) to simulate existing deteriorated concrete buildings. The compressive strength values obtained from testing three concrete cylinders of 150 mm (5.9 in.) diameter and 300 mm (11.81 in.) high for each column specimen are given in Table 1. The compressive strength of mortar was designed to be the same as that of concrete columns, and was measured to be 23.4 MPa (3.39 ksi).

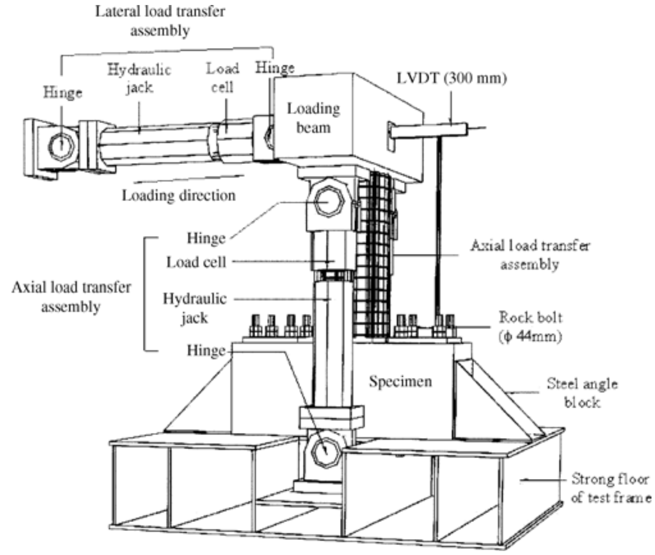

Fig. 2—Test setup. (Note: 1 mm = 0.0394 in.)

Table 2 shows the mechanical properties of the internal reinforcing bars, wire rope, steel plate, and eye-bolt used in the present study. The yield strength values of the 8 mm (0.31 in.) diameter internal hoop bar and eye-bolt were calculated using the 0.2% offset method.

Test procedure and instrumentation

All the specimens were tested under constant axial load and cyclic lateral loads in the steel test frame. The bottom stub of each column was fixed to the base of the test frame using eight steel rock bolts of 44 mm (1.73 in.) diameter penetrating the stub and two steel angles on both sides of the bottom stub as shown in Fig. 2, to achieve full fixity at the base. Axial compressive force was applied by pulling the load transfer assembly down using two 1000 kN (224.8 kips) capacity hydraulic jacks. After applying the full axial load, the specimen was coupled with a lateral load transfer assembly specially designed by reference to the test setup proposed by Ozcebe and Saatcioglu,[12] as illustrated in Fig. 2. After the final positioning of the specimen, lateral load reversals were applied at the center of the loading beam using a 1000 kN (224.8 kips) capacity hydraulic jack with a lateral displacement rate of 2 mm/min (0.079 in./min).

The specimens were subjected to the predetermined displacement history shown in Fig. 3. The magnitude of the lateral displacement at each cycle was dependent on the yield displacement Δ_y of each column. In the first cycle, approximately 75% of the predicted maximum lateral load $(V_u)_{pre}$ was applied in both the positive and negative directions. The prediction for the maximum lateral load of different columns was obtained by section analysis using the laminate method presented later in this paper. An experimental value for the yield displacement of each column was calculated by the extrapolation method specified in FEMA 356[13] as follows

$$\Delta_y = \frac{4/3 \times [|\Delta^+_{0.75}| + |\Delta^-_{0.75}|]}{2}$$

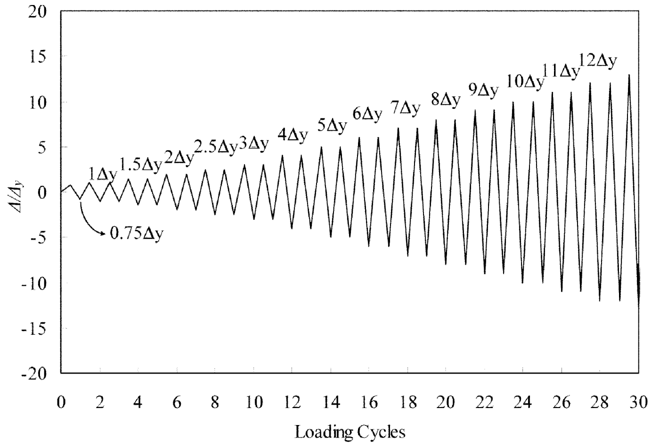

Fig. 3—Specified lateral displacement history.

where $\Delta^{+}_{0.75}$ and $\Delta^{-}_{0.75}$ indicate the lateral displacements measured at $0.75(V_u)_{pre}$ in the positive and negative directions, respectively.

Axial and lateral loads were measured by the load cells attached to the hydraulic jacks. Lateral displacement was recorded using 300 mm (11.81 in.) capacity linear variable differential transducers (LVDTs) mounted at the application point of the lateral load. In addition, strains in longitudinal reinforcement and T-shaped steel plates at various locations along the specimen length were recorded by 5 mm (0.19 in.) electrical resistance strain (ERS) gauges. The locations of the strain gauges on the longitudinal reinforcement and T-shaped steel plates are shown in Fig. 1(c).

TEST RESULTS AND DISCUSSIONS

Crack propagation and behavior of failure

The typical crack propagations and behaviors in failure of an unstrengthened column, a strengthened column without mortar cover, and a strengthened column with mortar cover are presented in Fig. 4. The failure mode for all test specimens was dominated by the flexural effect. Initial flexural cracks commonly appeared in the maximum-moment region for the first cycle of 0.75 times the yield displacement ($\Delta = 0.75\Delta_y$), and their length and number increased for the first cycle of $2\Delta_y$. After reaching the peak lateral load, different failure behavior was observed in the columns tested, depending on the strengthening technique used. For the unstrengthened column, spalling of the concrete cover started with severe flexural cracks at the first cycle of $1.5\Delta_y$, which corresponded approximately to the peak lateral load, and then the longitudinal reinforcement was severely buckled at the second cycle of $3\Delta_y$, as shown in Fig. 4(a). The failure of the unstrengthened column was accompanied by extensive buckling of the longitudinal reinforcing bars at the second cycle of $3\Delta_y$. On the other hand, no buckling of longitudinal reinforcement was observed in all the strengthened columns before $8\Delta_y$. In addition, the concrete of the strengthened column without mortar cover was generally crushed at the first cycle of $6\Delta_y$ after peak lateral load, that is, spalling of the concrete cover in the strengthened column was greatly delayed compared with the unstrengthened column. With the increase of lateral displacement of the column after crushing of concrete, the end of the T-shaped steel plates reached the bottom stub of the column and, as a result, buckling of the T-shaped steel plates occurred. The failure of the strengthened columns without mortar cover was dominated by buckling of the T-shaped steel plates and fracture of the longitudinal reinforcement at the large deformation of the column, as shown in Fig. 4(b). The T-shaped steel plates supported by pretensioned wire rope were quite effective in preventing spalling of the concrete cover and buckling of longitudinal reinforcement. For the strengthened columns with mortar cover, very few flexural cracks developed along the column length as the mortar under tensile stress separated from the column base with the increase of lateral displacement.

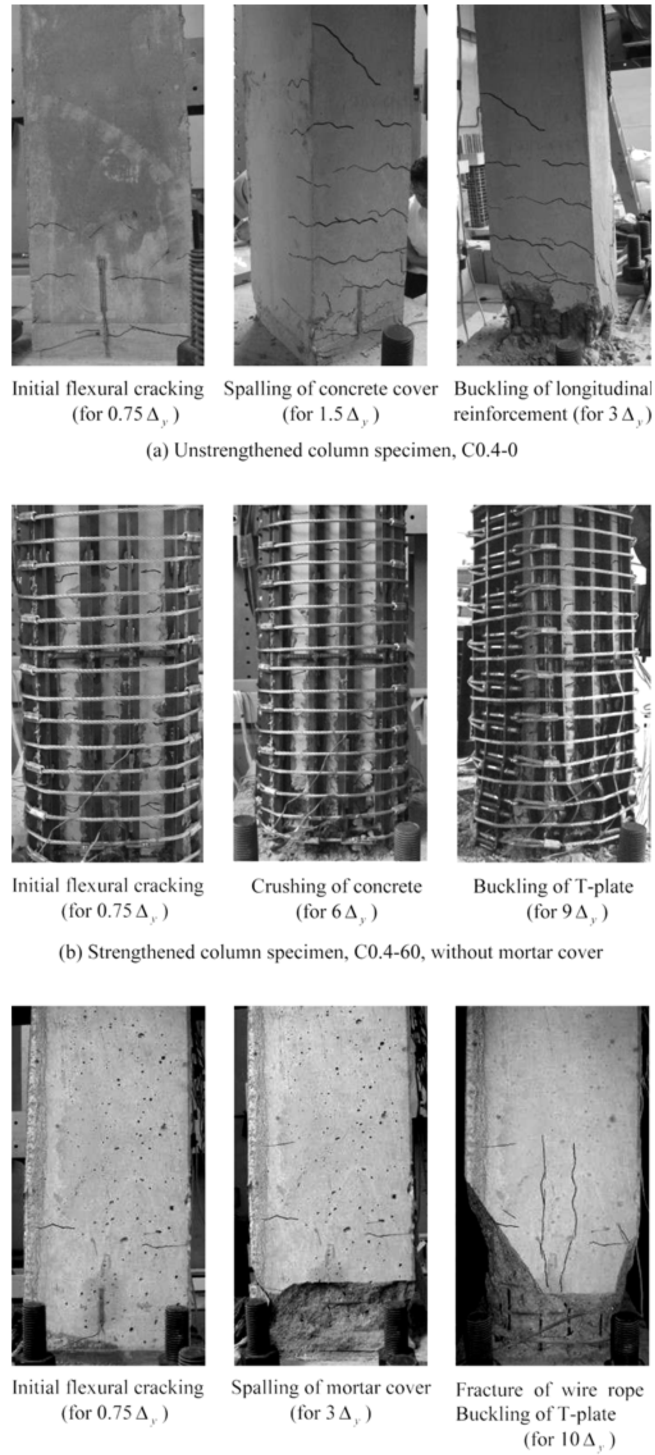

Fig. 4—Typical crack propagation and behavior of failure.

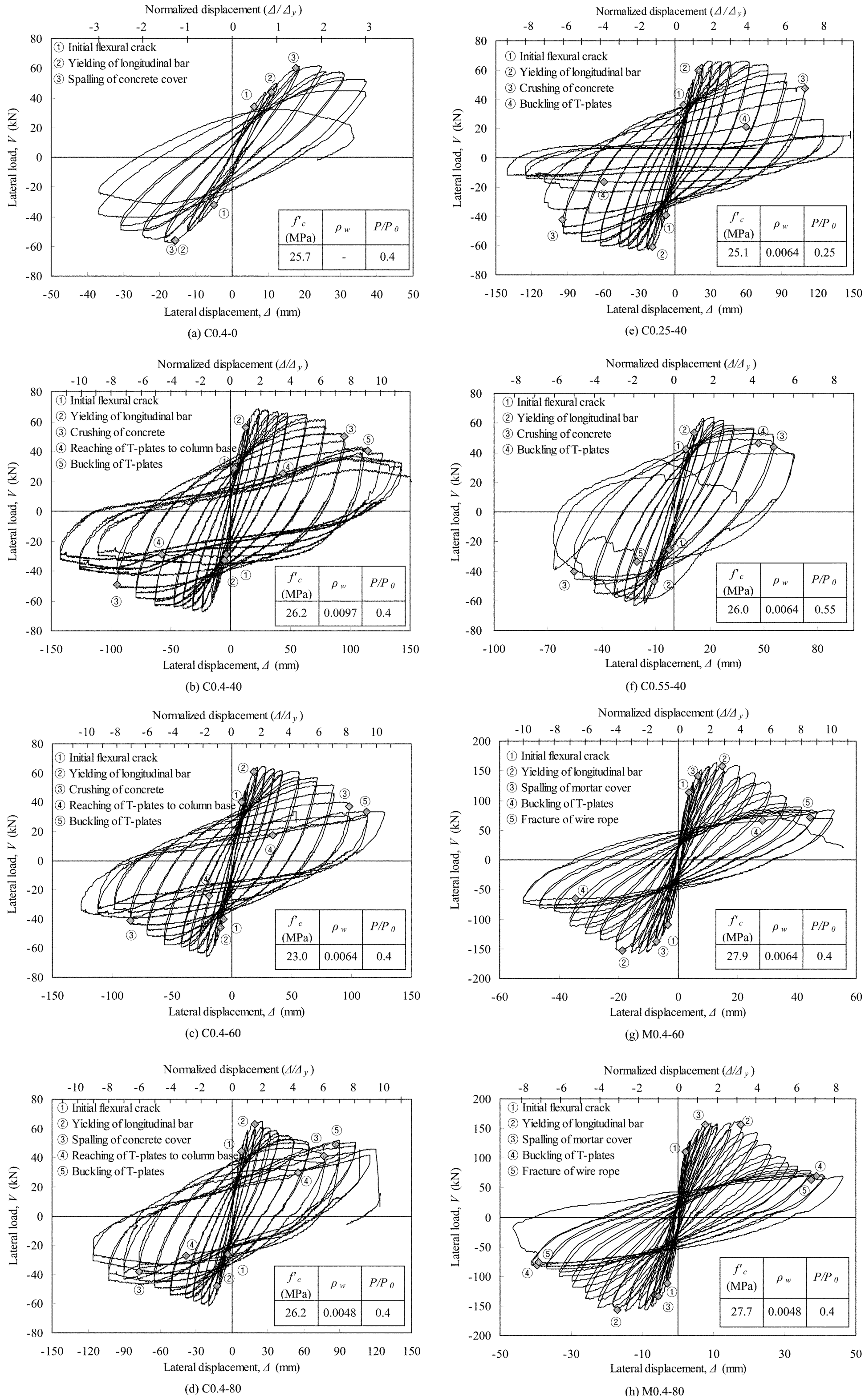

Fig. 5—Lateral load-displacement relationship. (Note: 1 kN = 0.2248 kips; 1 mm = 0.0394 in.)

After spalling of the mortar cover, vertical cracks also appeared along the T-shaped steel plates, as shown in Fig. 4(c). The buckling of the T-shaped steel plates and fracture of the wire rope dominated the failure of strengthened columns with mortar cover, in a way similar to strengthened columns without mortar cover.

Lateral load-displacement relationship

Figure 5 shows the lateral load-displacement relationship for different test specimens. Of the numerous properties suggested to quantitatively evaluate the ductility of the concrete columns, the authors used the member displacement ductility ratio μ_Δ (= Δ_{80}/Δ_y) and the work damage indicator

Table 3—Summary of test results and comparison with predictions

Specimen	Experimental results											Predicted V_n, kN		$(V_n)_{Exp.}/(V_n)_{Pre.}$	
	V_{cr}, kN		V_y, kN		V_n, kN										
	V_{cr}^+	V_{cr}^-	V_y^+	V_y^-	V_n^+	V_n^-	Average	Δ_y, mm	Δ_{80}, mm	μ_Δ	W_{80}	ACI 318-05	This study	ACI 318-05	This study
C0.4-0	34 $(0.75\Delta_y)$	32 $(0.75\Delta_y)$	44 $(1\Delta_y)$	56 $(1.5\Delta_y)$	61 $(1.5\Delta_y)$	56 $(1.5\Delta_y)$	58.5	12.3	32.5	2.64	5.2	51.9	53.4	1.13	1.10
C0.4-40	29 $(0.75\Delta_y)$	29 $(0.75\Delta_y)$	56 $(1\Delta_y)$	33 $(1.5\Delta_y)$	67 $(1.5\Delta_y)$	66 $(1.5\Delta_y)$	66.5	12.5	79.44	6.36	237.7	51.7	63.8	1.29	1.04
C0.4-60	40 $(0.75\Delta_y)$	40 $(0.75\Delta_y)$	61 $(1\Delta_y)$	46 $(1.5\Delta_y)$	64 $(2\Delta_y)$	65 $(2\Delta_y)$	64.5	12.06	70.83	5.87	199.6	48.6	57.0	1.33	1.13
C0.4-80	49 $(0.75\Delta_y)$	29 $(0.75\Delta_y)$	70 $(1\Delta_y)$	53 $(1.5\Delta_y)$	62 $(2\Delta_y)$	60 $(2\Delta_y)$	61.0	12.84	52.05	4.05	100.9	51.8	57.7	1.18	1.06
C0.25-60	36 $(0.75\Delta_y)$	39 $(0.75\Delta_y)$	60 $(1\Delta_y)$	61 $(1.5\Delta_y)$	65 $(2\Delta_y)$	62 $(2\Delta_y)$	63.5	15.62	93.96	6.02	302.9	51.8	56.7	1.23	1.12
C0.55-60	42 $(0.75\Delta_y)$	25 $(0.75\Delta_y)$	47 $(1\Delta_y)$	53 $(1.5\Delta_y)$	64 $(2\Delta_y)$	63 $(2\Delta_y)$	63.5	11.06	44.40	4.01	102.3	47.4	56.8	1.34	1.12
M0.4-60	113 $(0.75\Delta_y)$	110 $(0.75\Delta_y)$	158 $(3\Delta_y)$	156 $(3\Delta_y)$	164 $(2.5\Delta_y)$	158 $(2.5\Delta_y)$	161.0	5.2	26.04	5.01	75.2	160.6	163.2	1.00	0.99
M0.4-80	111 $(0.75\Delta_y)$	112 $(0.75\Delta_y)$	156 $(3\Delta_y)$	160 $(3\Delta_y)$	160 $(2.5\Delta_y)$	157 $(2.5\Delta_y)$	158.5	5.5	26.10	4.75	50.2	159.9	160.9	0.99	0.99

Note: V_{cr} is lateral load at which initial flexural crack occurred; V_y is lateral load at which longitudinal reinforcement yielded; V_n is peak lateral load; Δ_y is yield displacement of column as average of both loading directions; Δ_{80} is lateral displacement of column at $0.8V_u$ on descending branch of lateral load-displacement (V-Δ) curve, as average of both loading directions; μ_Δ is displacement ductility ratio; and W_{80} is work damage indicator.
Superscripts + and – refer to positive and negative loading directions, respectively.
Parentheses indicate V-Δ loop of incremental yield displacement at which specified features given in table occurred.
1 kN = 0.2248 kips; 1 mm = 0.0394 in.

W_{80} proposed by Sheikh and Khoury,[14] where the subscript 80 indicates the value corresponding to 80% of the ultimate strength $(0.8V_n)$ on the descending branch of the lateral load-displacement curve of the columns. The values of μ_Δ and W_{80} calculated from the lateral load-displacement of test specimens are summarized in Table 3.

Effect of amount of wire rope

The amount of wire rope had little influence on the initial stiffness and yielding of longitudinal reinforcement of the strengthened columns without mortar cover, as shown in Fig. 5(a) to (d) and Table 3. The longitudinal reinforcement of the strengthened columns without mortar cover and the unstrengthened column commonly yielded before reaching their peak lateral load. Strengthened columns without mortar cover had only 1.04 to 1.13 times higher flexural capacity than the unstrengthened column, showing that the confinement effect provided by the wire rope and the T-shaped steel plates caused only a slight increase in this property with increasing the volume ratio of wire rope ρ_w. The strengthening procedure was highly effective in enhancing the ductility of concrete columns, causing the flexural ductility of the strengthened columns to be much higher than that of the unstrengthened column. The μ_Δ and W_{80} of the strengthened column having ρ_w of 0.0048 were 1.53 and 19.4 times, respectively, as much as those of the unstrengthened column. In particular, the strengthened column having ρ_w of 0.0097 sustained approximately 80% of its ultimate flexural strength up to the second cycle of $6\Delta_y$. Both the stiffness degradation and strength reduction rate with every load cycle were also much lower for the strengthened columns compared with the unstrengthened column, indicating that the developed strengthening technique can provide excellent confinement in the concrete cover and core of a column, even at large deformations after the ultimate strength of the columns. Hence, the cyclic behavior of the strengthened columns could be improved with the increase of the amount of wire rope. The lateral load of strengthened columns dropped suddenly with either fracture of the longitudinal reinforcement or severe buckling of T-shaped steel plates, regardless of the amount of wire rope, as shown in Fig. 5(b) to (d).

Effect of axial load level

The axial load level P/P_0 significantly influenced the initial stiffness, yielding of longitudinal reinforcement, and ductility of strengthened columns without mortar cover, as shown in Fig. 5(c), (e), and (f) and Table 3. The initial stiffness of the strengthened columns increased with the increase of the axial load level and, therefore, Δ_y decreased. The strengthened columns having P/P_0 of 0.4 had a slightly higher flexural capacity than those having P/P_0 of 0.25 or 0.55. Therefore, it seems that P/P_0 to induce balanced failure in the columns strengthened with the developed procedures is approximately 0.4. On the other hand, substantial reductions in μ_Δ and W_{80} were observed with an increase in axial load from $0.25P_0$ to $0.55P_0$, in agreement with the observation that a higher axial load led to an increase in the rate of stiffness degradation with every load cycle and adversely affected the cyclic performance of strengthened columns. This trend is generally observed in tied columns.[10,14] However, the values of μ_Δ and W_{80} of strengthened Specimen C0.55-60 were 1.52 and 19.7 times, respectively, as much as those of the unstrengthened column, though the axial load level in the strengthened column was higher than in the unstrengthened column.

Effect of mortar cover for strengthening steel elements

Mortar cover was significantly effective in enhancing the initial stiffness and flexural capacity of the strengthened columns, as shown in Fig. 5(g) and (h). As mortar cover increases the section area of column, the strengthened columns with mortar cover commonly showed higher initial stiffness than the strengthened columns without mortar cover and unstrengthened column. The strengthened columns with mortar cover reached their ultimate strength shortly after spalling of the mortar cover. The flexural capacity of strengthened columns with mortar cover was at least 2.5 times higher than that of the comparable strengthened columns without mortar cover, as reported in Table 3. In addition, yielding of the longitudinal reinforcement occurred after the peak lateral load because of the reduction of the ratio of moment lever arm to section depth; this observation was different from that of strengthened columns without mortar cover. On the other hand, the ductility of the strengthened columns with mortar cover was inferior to that of comparable strengthened columns without mortar cover, showing that W_{80} of the strengthened column with mortar cover was approximately 50% lower than that of the comparable strengthened column without mortar cover, regardless of the amount of wire rope, as given in Table 3. In addition, a much higher strength reduction rate with every load cycle was observed for the strengthened columns with mortar cover than for the comparable strengthened columns without mortar cover.

Strain of T-plates and longitudinal reinforcement

Figure 6 shows the typical strain behavior of the T-shaped steel plate and longitudinal reinforcement in unstrengthened Specimen C0.4-0 and strengthened Specimens C0.4-60 and M0.4-60. The strains used to plot Fig. 6 were measured by ERS gauges located 125 mm (5 in.) from the interface between column and bottom stub in tensile zone of positive loading direction at the first cycle of every incremental yield displacement. The strains of the T-shaped steel plates of the strengthened columns were varied only between –200 μ and 100 μ up to $7\Delta_y$, regardless of the presence of mortar cover, indicating that a T-shaped steel plate not anchored fully into a column base cannot transfer the flexural loads. On the other hand, the strain measured from longitudinal reinforcement increased with the increase of the incremental lateral displacement of the column. In particular, similar strain behavior of longitudinal reinforcement was observed in both unstrengthened column and strengthened columns without mortar cover. The strain of longitudinal reinforcement in the strengthened column with mortar cover was generally lower than that in the strengthened column without mortar cover at the same level of incremental yield displacement.

Figures 7(a) and (b) show the typical strain distribution for longitudinal reinforcement recorded along the column length at the first cycle of each incremental yield displacement for Specimens C0.4-60 and M0.4-60, respectively. The normalized vertical axis of these figures indicates the ratio between the position of ERS gauges measured from the section of maximum moment L_{ERS} (in mm) and the length from the section of maximum moment to the point of contraflexure L_h (in mm). For the strengthened column without mortar cover, the first yielding of longitudinal reinforcement occurred at the maximum moment region for $1.5\Delta_y$, and the yielding section of longitudinal reinforcement widened with the increase of Δ_y. At the peak lateral load ($\Delta = 2\Delta_y$), the extent of the yielding section of longitudinal reinforcement was roughly equivalent to $0.2L_h$ from the critical section and increased up to approximately $0.27L_h$ for $6\Delta_y$, indicating that the extension of the yielding section of the longitudinal reinforcement after peak lateral load is very slow and small. The strain distributions of longitudinal reinforcement for the strengthened column with mortar cover was also similar to the trend observed for the strengthened column without mortar cover, except that for the latter, the first yielding occurred at $3\Delta_y$.

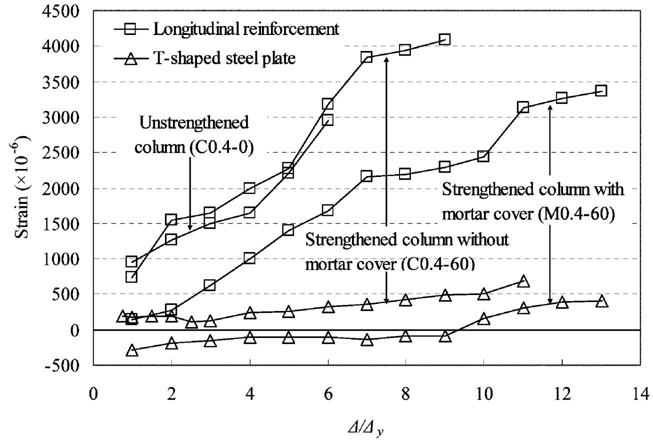

Fig. 6—Strain behavior of longitudinal reinforcement and T-plate in critical section.

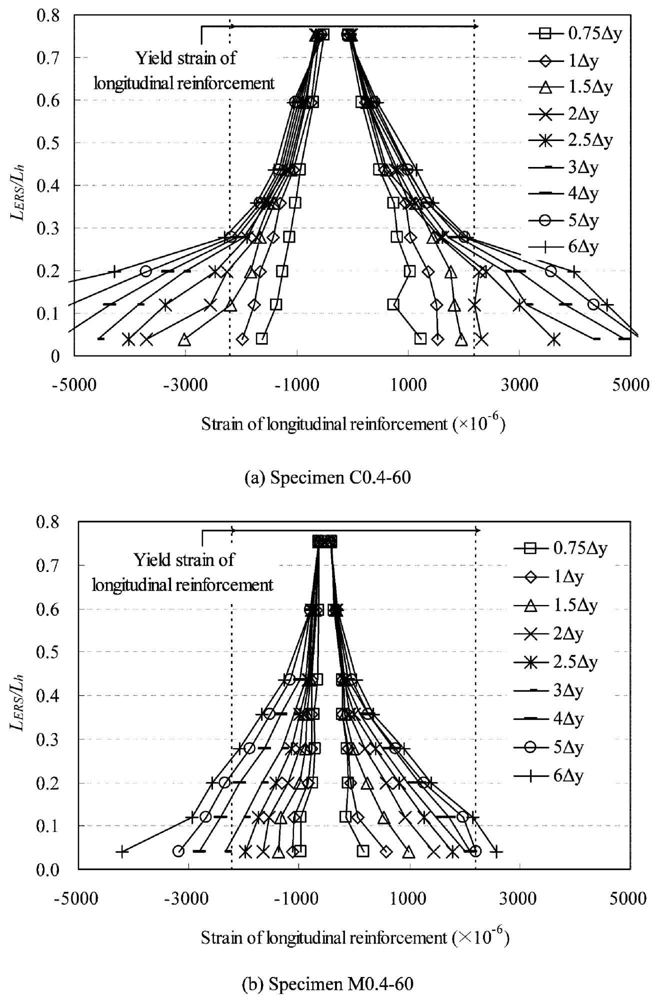

Fig. 7—Typical strain distribution of longitudinal reinforcement along strengthened column length.

PREDICTION OF LATERAL LOAD-DISPLACEMENT RELATIONSHIP

Stress-strain relationship of materials

Yang et al.[9] proposed the stress-strain characteristics of concrete confined by wire rope and T-shaped steel plate

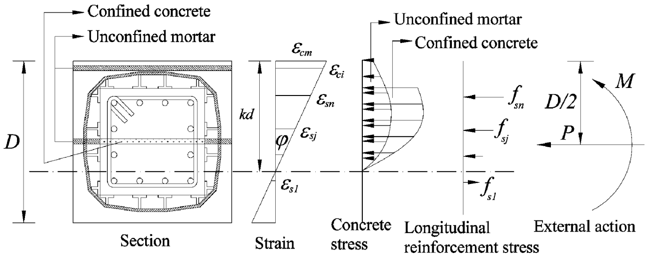

Fig. 8—Idealized distribution of strain and stress in strengthened column section.

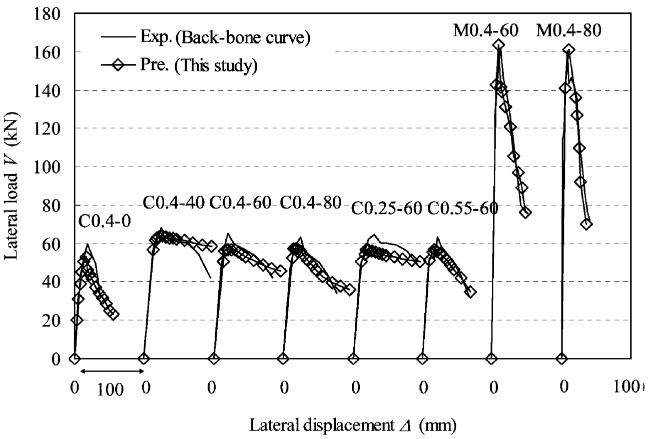

Fig. 9—Comparisons of measured and predicted monotonic lateral load-displacement relationship of column specimens. (Note: 1 kN = 0.2248 kips; 1 mm = 0.0394 in.)

units based on the equivalent uniform confinement concept and calibrated against test results of the strengthened columns subjected to concentric axial loads. A more detailed stress-strain model can be found in Reference 9. The stress-strain relationship of unconfined concrete is reproduced using the model proposed by Hognestad.[15] Tensile and compressive longitudinal reinforcing bars are also assumed to be elastic perfectly plastic material with yield strength f_y (in MPa) and elastic modulus E_s of 200 GPa (29,000 ksi).

Moment-curvature relationship

The laminae method[10] shown in Fig. 8 is highly useful for predicting the moment-curvature relationship simulating the section performance of a reinforced concrete member governed by flexure. Theoretical moment-curvature relationships for reinforced concrete sections strengthened with the developed procedures and subjected to combined flexure and axial load can be derived on the basis of the following assumptions: plane sections remain plane after bending; the tensile strength of concrete is neglected; unconfined cover concrete and mortar carry no stress at strains greater than 0.004; T-shaped steel plates with no anchorage into a column base contribute to confine concrete only; and stress-strain relationships of materials are given by the models presented in the previous section. The curvatures associated with a range of flexure and axial loads may be determined using these assumptions and from the requirements of strain compatibility and equilibrium of internal forces calculated using the stress-strain relationship of the different materials.[10] For the idealized section of the strengthened columns, therefore, the theoretical moment-curvature relationship for a given axial load level can be obtained by incrementing the concrete strain at the extreme compression fiber ε_{cm}.

Idealized curvature-displacement relationship

Each test column is idealized as a cantilever column. The elastic contribution to the displacement develops over the full length of the column and the inelastic displacement occurs at the plastic hinge formed in the critical section.[10] In addition, the plastic hinge rotation at the base can be assumed to be concentrated at the center of the plastic hinge, and the equivalent plastic hinge length l_p after ultimate strength of the column section can be considered to be constant. From the idealized distributions of curvature along the column length, therefore, displacement Δ at the free end of the column for each curvature φ at the critical section can be calculated from

$$\Delta = \frac{\varphi L_h^2}{3} \qquad \text{for } \varphi \le \varphi_y$$
$$\Delta = \frac{\varphi_y L_h^2}{3} + (\varphi - \varphi_y) l_p \left(L_h - \frac{l_p}{2}\right) \qquad \text{for } \varphi > \varphi_y \tag{1}$$

where φ_y indicates the curvature at the ultimate strength. From the moment distribution along the column length, lateral load V for each curvature can be also calculated by M/L_h, where M is moment calculated from the section laminae method.

The equivalent plastic hinge length l_p for reinforced concrete columns is still controversial and various empirical expressions have been proposed.[9,16] In the current analysis, a simpler expression proposed by Priestley and Park[16] is used as follows

$$l_p = 0.08L_h + 0.022d_b f_y \tag{2}$$

where d_b is the diameter of longitudinal reinforcement in mm. The equivalent plastic hinge length L_h for the column specimens calculated from Eq. (2) is $0.17L_h$, which is slightly lower than the values shown in Fig. 7.

COMPARISONS OF PREDICTIONS AND TEST RESULTS

Comparisons of predicted and measured monotonic lateral load-displacement curves of reinforced concrete columns strengthened with the developed procedures are shown in Fig. 9. Backbone curves obtained from the cyclic lateral load-displacement curves plotted in Fig. 5, using the routine specified in FEMA 356,[13] are used for comparisons. The peak lateral loads predicted from the current theoretical analysis and the equivalent stress block specified in ACI 318-05 are also given and compared with test results in Table 3. ACI 318-05 underestimates the flexural capacity of the strengthened columns without mortar cover, and the disagreement increases with the increase of the amount of wire rope and axial load level, because the confinement effect is not reflected in ACI 318-05. The peak lateral loads measured in the strengthened columns with mortar cover are very close to predictions obtained from ACI 318-05 and the current theoretical analysis. This may be attributed to the mortar cover not being placed monolithically against the column base, so that the applied flexural loads cannot be transferred completely. The average and standard deviation of the ratio between measured peak lateral load and predictions obtained from ACI 318-05 are 1.18 and 0.014, respectively. On the other hand, the predictions obtained from the current theoretical analysis show better agreement with test results, showing that the average and standard deviations of the ratio between experimental and analytical peak lateral loads are 1.07 and 0.06, respectively. Furthermore, the predicted descending branch of the lateral load-displacement curve is in good agreement with test results, regardless of the presence of the mortar cover, though the lateral displacement of the column specimens is calculated using the idealized curvature-displacement relationship.

CONCLUSIONS

The effect of confinement provided by wire rope and T-shaped steel plate units on the flexural behavior of concrete columns would be influenced by the size of column section and the ratio of area of cover mortar to that of column section. To ascertain this, therefore, it is necessary to collect experimental data on full-scale column specimens. Although the developed strengthening procedures were examined using the small-scale column specimens, the following conclusions are clearly drawn:

1. Wire rope and T-shaped steel plate units were highly effective in preventing spalling of concrete cover and buckling of longitudinal reinforcement.

2. The flexural capacity of strengthened columns without mortar cover was slightly higher than that of the unstrengthened column. The flexural ductility of strengthened columns, however, was much higher than that of the unstrengthened column, indicating that the displacement ductility ratio and the work damage indicators of the strengthened column having a volume ratio of wire rope of 0.0048 were 1.53 and 19.4 times, respectively—as much as those of the unstrengthened column.

3. The flexural capacity of the strengthened columns having an axial load level of 0.4 was slightly higher than that of the strengthened columns having axial load level of 0.25 or 0.55, indicating that the axial load level to induce balanced failure in the reinforced concrete columns strengthened with the developed procedures is approximately 0.4. On the other hand, an increase in axial load level substantially reduced the ductility of the strengthened columns.

4. The flexural capacity of strengthened columns with mortar cover was at least 2.5 times higher than that of the comparable strengthened columns without mortar cover. On the other hand, the work damage indicator of the strengthened column with mortar cover was approximately 50% lower than that of the comparable strengthened column without mortar cover, regardless of the amount of wire rope.

5. The equivalent plastic hinge length of the strengthened columns was measured as between 0.2 and 0.27 times the column length measured from the section of maximum moment to the point of contraflexure.

6. The lateral load-displacement relationship of the strengthened columns predicted from the proposed numerical analysis is in good agreement with test results.

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