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Confinement Effects of High-Strength Reinforced Concrete Tied Columns

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Abstract: An experimental study was conducted to investigate the effectiveness of transverse steel in reinforced concrete tied columns subjected to monotonically increasing axial compression. Eighteen large-scale columns (260 × 260 × 1,200 mm) were tested. Effects of such main variables as concrete compressive strength, configurations of transverse steel, transverse reinforcement ratio, spacing of transverse steel, and spalling of concrete cover were investigated. High-strength concrete columns under concentric axial loads show extremely brittle behavior unless the columns are confined with transverse steel that can provide sufficiently high lateral confinement pressure. A consistent decrease in the deformability of the column test specimens was observed with increasing concrete strength. Test results of this study were compared with existing confinement models of modified Kent-Park, Sheikh-Uzumeri, Mander, and Saatcioglu-Razvi. The comparison indicates many existing models to predict the behavior of confined concrete overestimate or underestimate the ductility of confined concrete.

Keywords: high strength concrete, tied columns, confinement, transverse steel, confined stress-strain relationship.

1. Introduction

Reinforced concrete structures are designed to behave ductile manner against the earthquake in general. Accordingly, the structural members are designed carefully in detail to manifest this desired behavior of ductility in advance. The details of transverse steel at the hinge region of the concrete column need a careful attention for the case of moment ductile frame structure. For the past several years, many detailed studies on how to improve the strength and ductility of the concrete structure through transverse steel have been actively pursued. As the result, it has been shown that the ductility of the concrete column is the most effective in proper confinement of the concrete core by transverse steel and the lateral support of the main longitudinal steel in transverse direction.

The most cited studies on the stress-strain characteristics of the concrete structure in consideration of the strength and ductility improvement of the concrete confined by transverse steels are listed as it follows. Kent-Park (1971) and Scott-Park (1982) among others have proposed a confinement model for the normal strength concrete each of their own. Later on, Sheikh-Uzumeri (1980), Mander et al. (1980), Saatcioglu-Razvi (1992) improved previously proposed models and suggested different models for the stress-strain characteristics of concrete using a variety of variables. However, some questions were raised against these models in that they were all based on normal strength concrete and have inherent safety issues in their application to high strength concrete. Up to now from the late 1980's, Fafitis-Shah (1985), Muguruma (1983, 1991), Cusson-Paultre (1995), Saatcioglu-Razvi (1999) have carried on the research on the stress-strain characteristics of high strength confined concrete.

Nevertheless, there are seldom any models to predict the property of high strength concrete accurately as of yet, and it has been reported that there is a shortage of data in this area of research.[4]

The technology for high strength concrete has improved remarkably during the past decade, and the practical advantages of using such characteristics of high strength concrete as the higher modulus of elasticity and the reduction of the cross section along with increased compressive strength of the concrete have attracted the interest of many researchers. Unfortunately, many countries are using high strength concrete in the absence of codes and regulations with respect to the safety of the structure. Moreover, the variables used to define the ACI Code provisions with respect to transverse (lateral) confinement were obtained from the results of the experiment on reinforced concrete member of the concrete compressive strength 400 kgf/cm^2 or below.[1] Additionally, it is the current situation that there is a shortage of data on the performance of large-scaled (minimum section of 200 mm or above) high strength reinforced concrete tied columns.

Thus, this study carried out a structural experiment on large scaled columns constructed with high strength concrete. The results of this experiment will be used to evaluate the confinement effect of high strength concrete columns and to identify the validity and shortcomings of previous confinement models through a comparison study with the experimental results. It is

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expected that this study will be used as a fundamental data to propose a better rational model in the future.

2. Previous confinement models

Among the confinement models having been proposed in the past, the most noteworthy models of Modified Kent-Park, Sheikh-Uzumeri, Mander and Saatcioglu-Razvi as illustrated in Fig. 1 are compared and analyzed against the experimental results of this study to investigate each model's characteristics, and the possibility of applying them to high strength concrete is examined.

2.1 Modified Kent-Park model

In 1982, Scott, Park and Priestley have proposed the modified Kent-Park model, which maintained the essential features of previous Kent-Park model and added the capacity to consider the increase in strength of the concrete by confinement. Scott, et al. reported that the increase in strain at maximum stress and maximum stress increased in proportion to the volumetric ratio and yield strength of transverse steel and as much as the strength enhancement coefficient, K, in inverse proportion to concrete strength. In other words, the modified Kent-Park model defines Kf'_c as the maximum stress of confined concrete and proposes the strain corresponding maximum stress is 0.002 K. Additionally, the modified model found that the inclination of the stress-descending part after the maximum stress changed with concrete strength and yield strength of, volumetric ratio and spacing of transverse steel and that the ductility increased more than the ductility proposed in the previous model as the result of increased strength of the confined concrete.

$$\text{For } \varepsilon_c \le 0.002K;\ f_c = Kf'_c\left[\frac{2\varepsilon_c}{0.002K} - \left(\frac{\varepsilon_c}{0.002K}\right)^2\right] \qquad (1)$$

$$\text{for } \varepsilon_c > 0.002K;\ f_c = Kf'_c\,[1 - Z_m(\varepsilon_c - 0.002K)] \ge 0.2Kf'_c \qquad (2)$$

$$K = 1 + \frac{\rho_s f_{yh}}{f'_c} \qquad (3)$$

$$Z_m = \frac{0.5}{\dfrac{3 + 0.29f'_c}{145f'_c - 1{,}000} + \dfrac{3}{4}\rho_s\sqrt{\dfrac{h''}{s_h}} - 0.002K} \qquad (4)$$

where, ε_c : compressive strain of the concrete,
f_c : compressive stress of the concrete (MPa),
f'_c : compressive strength of the standard cylinder (MPa),
f_{yh} : yield strength of transverse steel (MPa),
ρ_s : volumetric ratio of transverse steel to concrete core,
h'' : width of concrete core (mm),
s_h : space between the centers of transverse steel (mm).

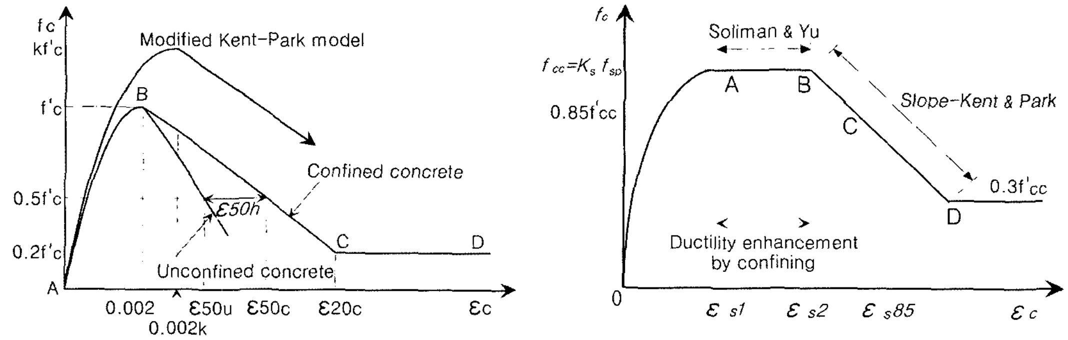
2.2 Sheikh-Uzumeri model

This is the model proposed by Sheikh and Uzumeri based on a theoretical method to compute effectively confined area from the geometric configuration of the cross section. Each of the coefficient used in this model was obtained from the regression analysis of the experimental result on rectangular tied columns with normal strength concrete. They are expressed as heuristic constants.

The unique characteristics of this model compared to modified Kent-Park model is that it reflects the configurations of transverse steel according to the distribution of main longitudinal steels.[2,7]

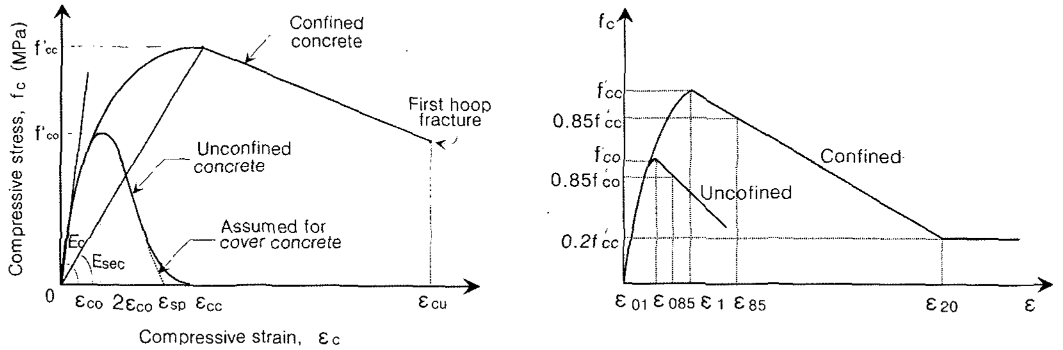
$$f'_{cc} = Ks \cdot f_{cp} \qquad (5)$$

where, $f_{cp} = 0.85f'_c$. The following equations are the characteristics of rectangular sectional columns.



(a) Modified Kent-Park model

(b) Sheikh-Uzumeri model



(c) Mander model

(d) Saatcioglu-Razvi model

Fig. 1 Previously proposed confined concrete model

$$K_s = 1 + \frac{1}{P_{occ}}\left(1 - \frac{nC^2}{\alpha B^2}\right)\left(1 - \frac{0.5s}{B}\tan\theta\right)^2 B^2 \beta(\rho_s f_s')^{\gamma} \tag{6}$$

$$P_{occ} = 0.85 f_c'(A_c - A_{st}) \tag{7}$$

$$\varepsilon_{sl} = 80K_s \cdot f_c' \cdot 10^{-6} \tag{8}$$

$$\frac{\varepsilon_{s2}}{\varepsilon_{oo}} = 1 + \frac{248}{C}\left[1 - 5\left(\frac{s}{B}\right)^2\right]\frac{\rho_s f_s'}{\sqrt{f_c'}} \tag{9}$$

$$Z = \frac{0.5}{\frac{3}{4}\rho_s\sqrt{\frac{B}{s}}} \tag{10}$$

$$\varepsilon_{s85} = \frac{0.15}{Z} + \varepsilon_{s2} \tag{11}$$

where, f_c : strength of confined concrete (MPa),
f_{cp} : strength of unconfined concrete (MPa),
B : width of concrete core as measured by the distance between the centers of transverse steel (mm),
c : distance of main longitudinal steel supported by transverse steel (cm),
s : distance between transverse steel (cm),
n : number of longitudinal steel supported by transverse steel,
ρ_s : volumetric ratio of transverse steel to concrete core,
f_c' : stress of transverse steel at maximum strength (MPa),
$\alpha, \beta, \gamma, \theta$: a constant to determine effectively confined area and confinement pressure.

2.3 Mander model

This model applied the concept of effectively confined area as proposed by Sheikh-Uzumeri to determine the effective lateral confinement pressure for a circular or rectangular sectional column from the effective confinement coefficient (k_e), which is the ratio of effectively confined area to core area. The effective lateral confinement pressure is considered to influence mainly on the strain at maximum stress and stress-descending part of the stress-strain curve of confined concrete. The stress-ascending part was computed by revised the equation proposed by Popovics in 1973, and ε_{cc}, having the greatest influence in the proposed model, was obtained by applying the "Five-Parameters" model, which William-Warnke proposed for the analysis of fractured plain at the three axial state in 1975.[6]

$$f_c = \frac{f_{cc}' x r}{r - 1 + x^r},\ x = \frac{\varepsilon_c}{\varepsilon_{cc}} \tag{12}$$

where, f_{cc}' : maximum strength of confined concrete (MPa),
ε_c : axial compressive strain of the concrete,
$\varepsilon_{cc} : \varepsilon_{co}\left[1 + 5\left(\frac{f_{cc}'}{f_{co}'} - 1\right)\right]$,
r : $E_c / (E_c - E_{\sec})$
E_c : $5{,}000\sqrt{f_{co}'}$
$E_{\sec}$: $f_{cc}' / \varepsilon_{cc}$

The ultimate strength is expressed as follows by computing each of the five parameters of William-Warnke's "Five-Parameters Model" from three axial state experiment of Schickert and Winkler in 1977.

$$f_{cc}' = f_{co}'\left(-1.254 + 2.254\sqrt{1 + \frac{7.794 f_l'}{f_{co}'}} - 2\frac{f_l'}{f_{co}'}\right) \tag{13}$$

with respect to rectangular sectional columns ;

$$f_l' = \frac{1}{2}k_e \rho_s f_{yh} \tag{14}$$

$$k_e = \frac{A_e}{A_{cc}} = \frac{\left(1 - \sum_{i=1}^{n}\frac{(w_i')^2}{6b_c d_c}\right)\left(1 - \frac{s'}{2b_c}\right)\left(1 - \frac{s'}{2d_c}\right)}{(1 - \rho_{cc})} \tag{15}$$

where, ρ_s : volumetric ratio of transverse steel to concrete core,
A_{cc} : cross sectional area of concrete core (cm^2),
A_e : effectively confined area at critical cross section (cm^2),
w_i' : the closest space between main longitudinal steel supported by transverse steel (cm)
s' : the closest (lengthwise) space between transverse steel in longitudinal direction (cm)
b_c : width of concrete core (cm)
d_c : depth of concrete core (cm)
n : number of main longitudinal steel
ρ_{ss} : cross sectional area ratio of main longitudinal steel to concrete core area

2.4 Saatcioglu-Razvi model

This model revised the model proposed by themselves previously as a confinement model for high strength concrete so that it could be applied as a general model for all range of concrete strength. It is a model proposed recently, and it can compute effective equivalent confinement pressure from the geometric configuration of the arrangement of main longitudinal steel in concrete column to define its stress-strain characteristics. This model requires complex computational procedures compared to other models. However, it has the feature to consider a variety of variables of influence on the confinement effect.[5,9]

stress-ascending part ;

$$f_c = \frac{f_{cc}'\frac{\varepsilon_c}{\varepsilon_1}r}{r - 1 + \left(\frac{\varepsilon_c}{\varepsilon_1}\right)^r} \tag{16}$$

where, r : $E_c / (E_c - E_{\sec})$,
$E_{\sec}$: f_{cc}' / ε_1(MPa),
ε_c : strain of confined concrete,
ε_1 : strain at the maximum strength of confined concrete.

stress-ascending part ; the straight line connecting and

$$\varepsilon_1 = \varepsilon_{01}(1 + 5k_3 K) \tag{17}$$

$$\varepsilon_{85} = 260 k_3 \rho_c \varepsilon_1[1 + 0.5k_2(k_4 - 1)] + \varepsilon_{085} \tag{18}$$

where, k_3 : $40 / f_{co}'$,
k_4 : $f_{yt} / 500$,
ρ_c : volumetric ratio of transverse steel,
K : $k_1 f_{le} / f_{co}'$,
ε_{085} : strain corresponding $0.85 f_c'$.

$$f_{cc}' = f_{co}' + k_1 f_{le} \tag{19}$$

$$k_1 = 6.7(f_{le})^{-0.17} \tag{20}$$

$$f_{le} = k_2 f_l \tag{21}$$

$$f_l = \frac{\sum A_s f_s \sin\alpha}{s b_c} \tag{22}$$

$$k_2 = 0.15\sqrt{\left(\frac{b_c}{s}\right)\left(\frac{b_c}{s_l}\right)} \leq 1.0 \tag{23}$$

where, f_{cc}' : strength of confined concrete (MPa),

f_{co}' : strength of unconfined concrete (MPa),

f_l : lateral confinement pressure (MPa),

f_{le} : equivalent lateral confinement pressure (MPa),

A_s : cross sectional area of transverse steel (cm^2),

f_s : strain of transverse steel at the maximum strength of confined concrete (MPa),

α : the angle between transverse steel and vertical cross section b_c,

b_c : distance between the centers of perimeter transverse steel (cm),

s : space between the centers of transverse steel (cm).

3. Experimental plan and method

3.1 Experimental plan

The test specimen had the size dimension of 260 × 260 × 1,200 mm, and its center part of 800 mm was set as the test region. Both ends of the specimen of 200 mm length was reinforced with transverse steel twice the quantity of transverse steel in the test region to prevent the local failure at the ends and to induce the failure at the test region. Additionally, both ends of the specimen were reinforced with carbon fibers in two layers after curing of the concrete. The thickness of the concrete cover was 20 mm, and the cross sectional area ratio of core to the column (A_c/ A_g)was set at 0.72 except for the specimens without the cover (NS).

Excepting D-typed cross-ties, all transverse steels had the hook angle of 135° and extended length of 6d$_b$, and they were anchored to the concrete core. The volumetric ratio of main longitudinal steels were kept almost the same at 2.25~2.36%. The variables of concrete strength (225~500 kgf/cm^2), volumetric ratio of transverse steel (according to the ACI Code of 80, 100, 120%), space between transverse steels (40~150 mm), arrangements of the main longitudinal steels (cf. Fig. 2) were considered to investigate their influence on the behavior of confined concrete. The details and list of specimens are presented in Fig. 2 and Table 1, respectively.

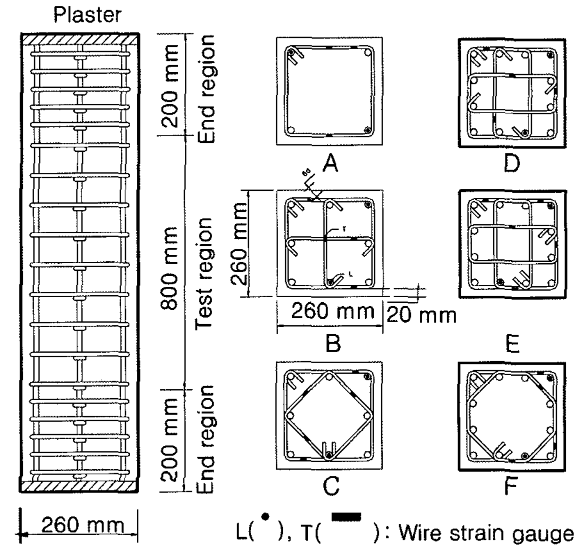


Fig. 2 Details of column specimens.

Table 1 Column properties.

Specimens	f_{ck} (kgf/cm^2)	Longitudinal reinforcement			Transverse reinforcement				
		Diameter (mm)	Ratio (%)	f_y (kgf/cm^2)	Diameter (mm)	Spacing (mm)	Volumetric ratio (%)	f_{yh} (kgf/cm^2)	$\frac{\rho_{prov.}}{\rho_{ACI}}$
NSC-P	245	-	-	-	-	-	-	-	-
NSC-A-10	260	4-D22	2.29	3,300	D10	90	1.543	4,450	1.0
NCS-B-10	245	8-D16	2.36	3,630	D8	100	1.302	5,100	1.0
NSC-D-10	265	12-D13	2.25	3,860	D8	130	1.335	5,100	1.0
NSC-E-10	225	12-D13	2.25	3,860	D8	150	1.350	5,100	1.0
NSC-F-10	255	12-D13	2.25	3,860	D8	120	1.305	5,100	1.0
NSC-E-NS	230	12-D13	2.65	3,860	D8	150	1.350	5,100	1.0
HSC-P	470	-	-	-	-	-	-	-	-
HSC-A-10	445	4-D22	2.29	3,300	D8	40	2.169	5,100	1.0
HSC-E-10	450	12-D13	2.25	3,860	D8	90	2.250	5,100	1.0
HSC-F-10	490	12-D13	2.25	3,860	D8	70	2.237	5,100	1.0
HSC-B-NS*	470	8-D16	2.76	3,630	D8	60	2.169	5,100	1.0
HSC-A-08	440	4-D22	2.29	3,300	D8	50	1.736	5,100	0.8
HSC-B-08	495	8-D16	2.36	3,630	D8	75	1.736	5,100	0.8
HSC-E-08	445	12-D13	2.25	3,860	D8	115	1.761	5,100	0.8
HSC-A-12	470	4-D22	2.29	3,300	D10	45	3.085	4,450	1.2
HSC-B-12	480	8-D16	2.36	3,630	D10	70	2.975	4,450	1.2
HSC-E-12	500	12-D13	2.25	3,860	D8	75	2.700	5,100	1.2

Denotation : NSC-A-10 ; First term-compressive strength of concrete, second term-configurations of transverse reinforcement(Fig. 2),
Third term-$\rho_{prov.}/\rho_{ACI}$ (ρ_{prov} : Volumetric ratios of transverse reinforcement designed in this study)
* NS : Specimens without shell(cover) concrete

3.2 Experimental equipment and method

The experiment used U.T.M. of 700 tonf capacity. The load was exerted concentric uni-axially at the center of the specimen by displacement control method, and the loading speed was planned in three stages to acquire the data by each specimen type in a reasonable manner. The load was exerted between $4\times10^{-6}\sim2\times10^{-5}$/sec of strain rate.

LVDTs were installed on embedded rods with four sides prior to the filling of the concrete to measure the axial strain of concrete core in the test region. Additionally, LVDTs were installed on both sides of the specimens entirely throughout the length to measure the displacement of the specimens in the axial direction. Fig. 3 shows the set-up of the column specimen.

4. Experimental result and analysis

4.1 Failure modes

The pattern of cracking and its progress was similar for all test specimens. A vertical crack started at the loading end or bottom end initially, and the progress of crack showed the tendency to develop to the test region.

Additionally, yielding of main longitudinal steels and spalling of cover concrete started just prior to and after maximum loading. Soon after, buckling of main longitudinal steels and opening of the hook on the transverse steel started gradually to cause the complete spalling of the cover. The buckling of main longitudinal steels was more severe for high strength concrete specimens (HSC) than for normal strength concrete specimens (NSC). As the space in the transverse steel was wider, the buckling occurred earlier and was more severe around the corner than inside the main longitudinal steel. Additionally, the degree of opening of the transverse steel hook was more severe for HSC than for NSC.

The yield of transverse steel took place at the phase of reducing load after the maximum load, and fracture of the transverse steel was observed in some HSC specimens. The mode of the fracture was at the intermediate hoop of C and F-type specimens. Failure surface was formed for almost all specimens, and it was more conspicuous for HSC specimen than for NSC specimen. In addition, the angle formed by horizontal section and failure surface was between the maximum of 90° (for concrete specimens without transverse steel) and the minimum of 25°. The angle was wider for HSC specimen than for NSC specimen and with the increase in the space between the transverse steel. Of particular notice was that one or two transverse steel(s) went through the center of failure surface in most cases. The damage due to crushing of confined concrete (concrete core) was observed at the failure surface to be more severe for HSC specimens than for NSC specimens. This is deemed to be due to its close relationship with the space between transverse steel. The crushing of concrete core was more severe at the corner compared to its inside.

4.2 Axial load carried by the concrete and analysis of concrete core behavior

The strain carried by concrete (P_{CONC}) was computed by subtracting the axial load carried by main longitudinal steel (P_{ST}) from the total axial strain of the column specimen (P_{TEST}) as measured and recorded during the experiment for each of the test specimen as shown in Fig. 4(a). The axial load carried by main longitudinal steel was calculated from total cross sectional area of the main longitudinal steel and the strain as measured by a gauge attached to the main longitudinal steel.

$$P_{CONC}=P_{TEST}-P_{ST} \tag{24}$$

The load-strain relationship of the confined concrete core was analyzed from the fact that the axial load carried by the concrete was shared by the concrete core and cover (shell) concrete as shown in Fig. 4(b). The analysis method is based on the load-strain relationship for the entire concrete column and the shared axial strain behavior of the concrete core and cover concrete as aforementioned. From Fig. 4(b), P_{oc} and P_{occ} are calculated as in the following eqs. (25) and (26).

$$P_{oc}=0.85f_{ck}(A_g-A_{st}) \tag{25}$$

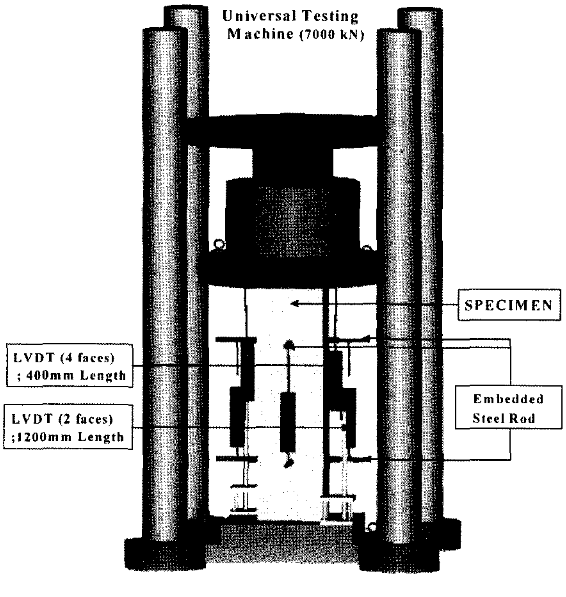
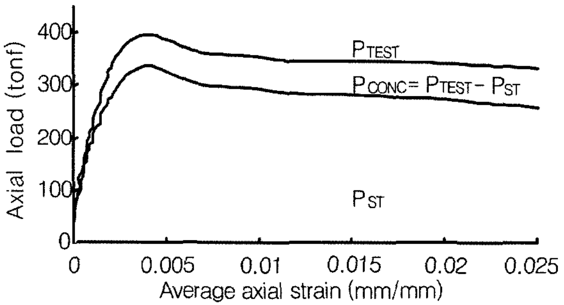


Fig. 3 Set-up of column specimen.



(a) Contribution of concrete

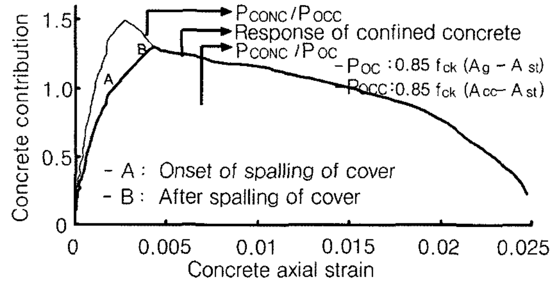


Fig. 4 Strain-load behavior of confined concrete.

$$P_{occ} = 0.85 f_{ck}(A_{cc} - A_{st}) \qquad (26)$$

where, f_{ck} : strength of concrete standard cylinder (kgf/cm^2),

A_g : total cross sectional area of the concrete column (cm^2),

A_{st} : cross sectional area of main longitudinal steel (cm^2),

A_{cc}: cross sectional area of concrete core (cm^2).

The result of the experiment on the confined concrete is analyzed as in the above and is shown in Table 2.

4.3 Behavior of confined concrete influenced by main variables

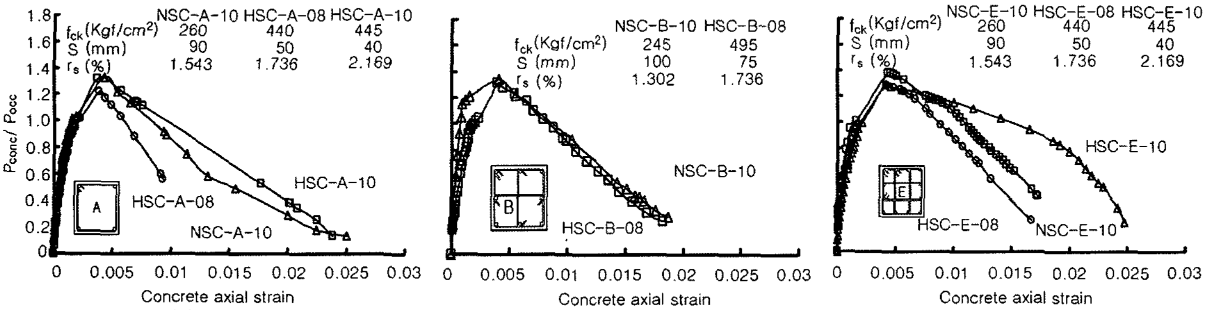
4.3.1 Effect of concrete strength

Because high strength concrete exhibits less lateral expansion under compression condition due to higher modulus of elasticity and less inside micro-crack compared to normal strength concrete, the lateral confinement pressure by the transverse steels is reduced. Fig. 5 shows the behavior of confined concrete in response to various concrete strength. The test specimens, NSC-A-10, HSC-A-08, and HSC-A-10 exhibited the strength enhancement in concrete by 1.33, 1.22, and 1.32, respectively, and also the enhancement in ductility by 7.56, 4.44, and 7.47, respectively. Similarly, the test specimens, NSC-E-10, HSC-E-08, and HSC-E-10 exhibited the strength enhancement in concrete by 1.38, 1.27, and 1.44, respectively, and also the enhancement in ductility by 8.02, 5.92, and 9.62, respectively. The experimental result attests for the fact that the concrete tied columns can obtain significant enhancement in strength and improvement in ductility by the lateral confinement pressure. Nevertheless, it can be also realized that the stronger confinement pressure is required to obtain the desired effect of similar enhancement in strength and ductility improvement as the concrete strength is increased. Moreover, it was found out that, given the same type (configuration) of transverse steel, the volumetric ratio of transverse steel for high strength concrete must be 50% or higher to attain the ductility of normal strength concrete.

Table 2 Test results.

Specimens	Axial load				Axial strain					Toughness
	P_{max} 1)(tf)	P_{cmax} 2)(tf)	P_{cmax}/P_{oc} 3)	P_{cmax}/P_{occ} 4)	ε_{max} 5)	ε_{cc} 6)	ε_{c85c} 7)	ε_{c50c} 8)	$\varepsilon_{c50c}/\varepsilon_{co}$ 9)	A_{50c}/A_{co} 10)
NSC-P	143.09	143.09	1.02	1.02	0.00208	0.00174	0.0028	0.0044	2.12	2.77
NSC-A-10	207.24	137.98	0.95	1.33	0.00242	0.00430	0.0065	0.0123	5.91	10.59
NCS-B-10	225.96	130.95	0.95	1.34	0.00226	0.00410	0.0066	0.0128	6.15	11.07
NSC-D-10	211.89	147.12	0.99	1.39	0.00256	0.00450	0.0083	0.0149	7.16	13.50
NSC-E-10	186.64	123.90	0.98	1.38	0.00261	0.00440	0.0085	0.0144	6.92	11.56
NSC-F-10	221.50	140.91	0.98	1.39	0.00250	0.00440	0.0071	0.0163	7.84	13.99
NSC-E-NS	171.90	111.85	1.22	1.22	0.00443	0.00360	0.0067	0.0172	8.27	11.59
HSC-P	269.43	269.43	1.00	1.00	0.00238	0.00223	0.0027	0.0037	1.55	1.91
HSC-A-10	293.70	233.56	0.94	1.32	0.00201	0.00370	0.0076	0.0155	6.51	10.10
HSC-E-10	351.16	258.03	1.02	1.44	0.00275	0.00414	0.0118	0.0214	8.99	16.04
HSC-F-10	364.22	278.60	1.01	1.43	0.00316	0.00469	0.0184	0.0193	8.11	16.21
HSC-B-NS	298.49	237.72	1.27	1.27	0.00362	0.00360	0.0139	0.0193	8.11	14.63
HSC-A-08	301.26	213.48	0.86	1.22	0.00224	0.00480	0.0057	0.0091	3.82	5.42
HSC-B-08	356.66	251.08	0.90	1.27	0.00270	0.00440	0.0070	0.0125	5.25	9.02
HSC-E-08	325.20	228.73	0.92	1.29	0.00272	0.00422	0.0077	0.0126	5.29	8.33
HSC-A-12	335.54	263.84	1.00	1.41	0.00267	0.00480	0.0103	0.0231	9.71	16.49
HSC-B-12	352.33	291.55	1.08	1.53	0.00370	0.00390	0.0172	0.0333	13.99	26.91
HSC-E-12	396.01	333.83	1.19	1.68	0.00419	0.00430	0.0184	0.0322	13.53	31.73

1)Max. axial load carried by column, 2)Max. axial load carried by concrete(Fig. 4), 3)P_{oc} : Axial capacity of total concrete cross section(eq. (25)), 4)P_{occ} : Axial capacity of concrete core(eq. (26)), 5)Axial strain corresponding to Max. load, 6)Axial strain in confined concrete corresponding to f'_{cc}, 7)Axial strain in confined concrete corresponding to $0.85f'_{cc}$, 8)Axial strain in confined concrete corresponding to $0.5f'_{cc}$, 9)ε_{co} : Axial strain in plain concrete corresponding to f_{ck}, 10)A_{50c} : Area under stress-strain curve of the confined concrete up to ε_{c50c}, A_{co} : Area under stress-strain curve of the unconfined concrete up to ε_{co}



(a) A-type (b) B-type (c) E-type

Fig. 5 Effects of concrete strength.

4.3.2 Effect of volumetric ratio of transverse steel

The transverse confinement pressure exerted on the concrete core is directly related to the quantity of transverse reinforcing steels. Fig. 6 shows the behavior of confined concrete by the volumetric ratio of the transverse steel. When the volumetric ratio of transverse steel was increased by about 70% in comparison of the test specimens HSC-A-08 to HSC-A-12, HSC-B-8 to HSC-B-12, and HSC-E-08 to HSC-E-12, the strength enhancement was 16%, 20%, and 30%, respectively, and the ductility impro-vement was about 2.8, 2.7, and 2.4 fold, respectively. From the test results, it is found that the increase in volumetric ratio of the transverse steel can directly enhance the strength of the confined concrete and also simultaneously improve the ductility of the confined concrete. Moreover, it is seen that the effective configu-ration of the transverse steel can exert a greater effect in the concrete strength.

4.3.3 Effect of spacing and configuration of transverse steel

The area of effectively confined concrete and buckling of main longitudinal steel are determined by the type (configuration) and spacing of transverse steel. Fig. 7 shows the behavior of concrete confined by the configuration and spacing of the transverse steels. As seen in Fig. 7(a), the strength and ductility of specimens with transverse steel of A-typed configuration had inferior strength and ductility than specimens of other transverse steel type (configuration). Additionally, it is shown that the type of transverse steel configuration influences the ductility enhancement greater than the strength enhancement. This means that the type of transverse steel configuration has greater confinement effect, given similar volumetric ratios of the laterally reinforced concrete tied columns. Comparing Figs. 7(a) and (b), the effect of transverse confinement type was greater for high strength concrete than for normal strength concrete. However, the behavior of the confined concrete specimens, which has the spacing between transverse steel equal to or over the half of the cross sectional width of the concrete core, was not much different from each other regardless of the type of the transverse steel configuration. This signifies the confinement effect limited by the spacing of the transverse steel as it has been previously reported.[5] Furthermore, it indicates that the spacing of transverse steel is yet another variable of influence to the behavior of confined concrete.

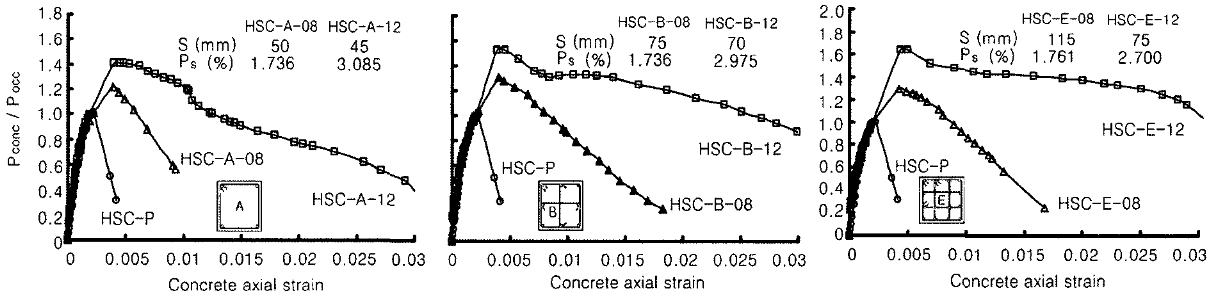
Thus, it is then construed that the densely-packed spacing of transverse steel can result in more efficient confinement effect. As the result, the influence of independent variables on the strength and ductility of confined concrete varies with the degree of confinement determined by other variables. Thus, when analyzing the behavior of confined concrete as a function of independent variables, it is construed that the analysis result should be limited to specific influence depending on the context of the test environmental setting and that these variables have very complex relationship of mutual dependency to each other (i.e. the interaction effect).

5. Comparison analysis of previous models with experimental results

5.1 Comparison of stress-strain relationship for confined concrete

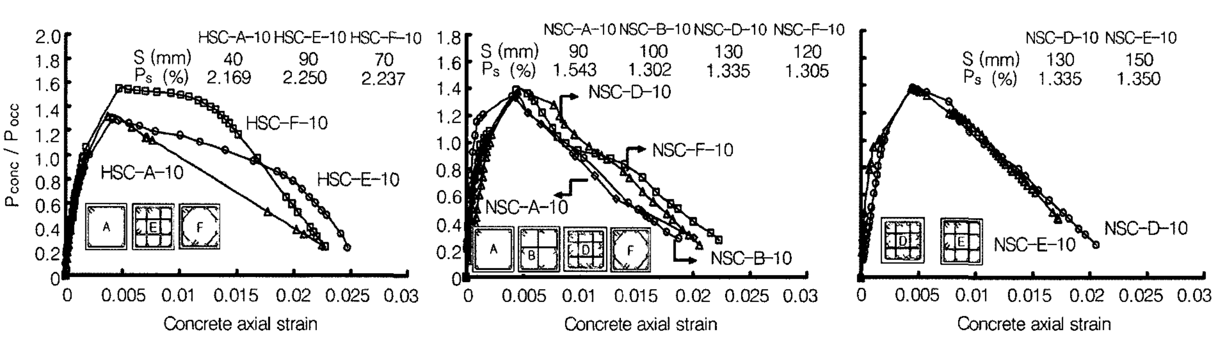
5.1.1 Modified Kent-Park model

As seen in Fig. 8, the modified Kent-Park model overestimates the maximum stress (f_{cc}) of confined concrete and underestimates the strain (ε_c) at the maximum stress. After the maximum stress, the behavior of confined model is significantly overestimated as the spacing of transverse steel becomes greater,



(a) A-type (b) B-type (c) E-type

Fig. 6 Effects of volumetric ratio of transverse reinforcement.



(a) HSC series (b) NSC series (c) NSC series

Fig. 7 Effects of spacing and arrangement of transverse reinforcement.

showing that the modified model is affected by the spacing of transverse steel. This is reasoned by the fact that the coefficient of strength increase, K, which is the main variable and explains the characteristics of this model the best, depends mainly on the volumetric ratio and that this model is a revised equation based on the result of experiments limited by experimental constraints.

5.1.2 Sheikh-Uzumeri model

As seen in Fig. 8, Sheikh-Uzumeri model predicts the maximum stress comparatively well, but it underestimates the strain at maximum stress. Additionally, it can be seen that the behavior varies significantly as the spacing of transverse steel is larger for the case of NSC specimen and as the volumetric ratio and the configuration type of the transverse steel change for the case of HSC specimen. This is because this model was proposed based on NSC type specimens and did not consider a variety of the volumetric ratio and the configuration type of the transverse steel fully due to its nature of experiments performed under limited scope. Moreover, it seems that the model focused on the effect of the configuration type of the transverse steel more than the spacing of the transverse steel. Additionally, it tends to overestimate the confinement force by assuming the stress of transverse steel at maximum strength (f_s) as the yield strength for the transverse steel.

5.1.3 Mander model

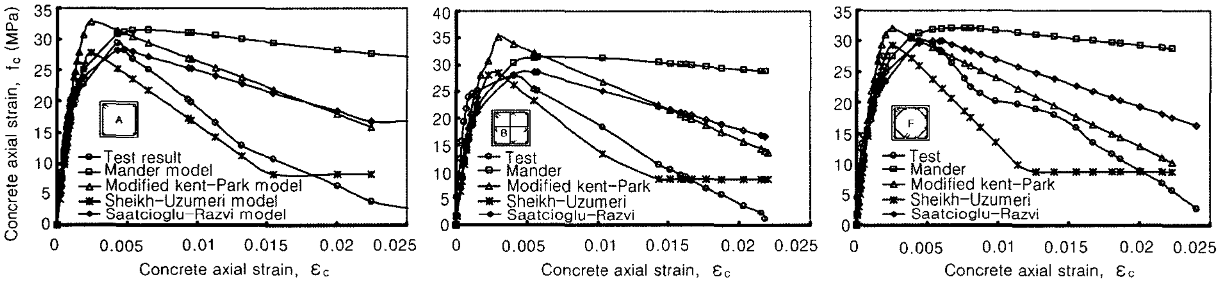
The Mander model predicts the stress ascending part comparatively well, but it overestimates the maximum strength and the strain (ε_{cc}) at maximum strength (f_{cc}). Additionally, as seen in Fig. 8, the stress descending part is overestimated compared to the experimental results of this study or other models. This is construed due to the fact that the Mander model unlike other models proposes the stress-strain relationship as one curve. Nevertheless, the Mander model predicts the experimental results reasonably well when the test specimens of compound type (E and F-type) of transverse steel configuration with high volumetric ratio are tested (cf. Fig. 8(i)).

5.1.4 Saatcioglu-Razvi model

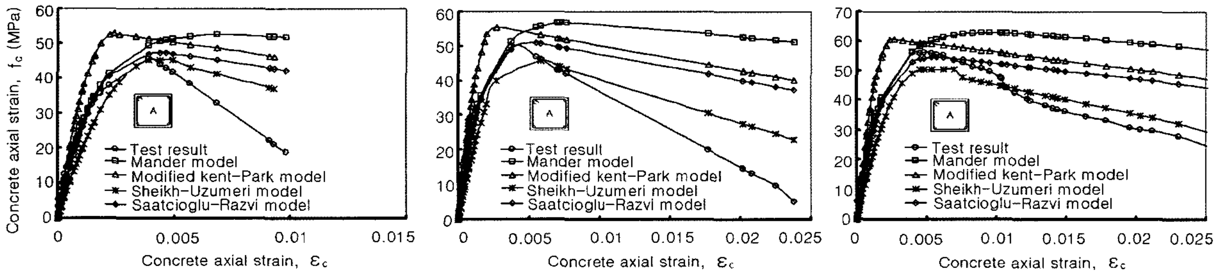
Saatcioglu-Razvi model predicts the ascending part of stress-strain relationship of confined concrete and the strain at maximum stress as well as the maximum stress relatively well. However, the descending part of the stress-strain relationship is conspicuously overestimated, and this tendency is accentuated for the case of specimens not having the auxiliary transverse steel (cross tie or intermediate hoop). It is reasoned that the narrow space between transverse steels has been overestimated in evaluating the transverse confinement pressure, requiring a more rational evaluation.

5.2 Comparison of strength and ductility of confined concrete

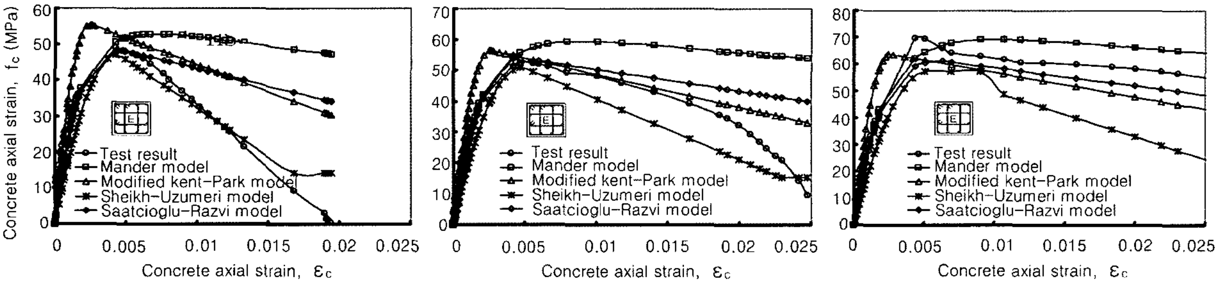
Fig. 9 shows that all models predict the strength of NSC confined concrete specimens relatively well but underestimate or



(a) NSC-A-10 (b) NSC-B-10 (c) NSC-E-10



(d) HSC-A-08 (e) HSC-A-10 (f) HSC-A-12



(g) HSC-E-08 (h) HSC-E-10 (i) HSC-E-12

Fig. 8 Comparison of previous model with test results.

overestimate the strength of HSC confined concrete specimens as the strength of the concrete is increased. This can be explained by the fact that modified Kent-Park, Sheikh-Uzumeri, and Mander models are based on the results of experiment on normal strength concrete and Saatcioglu-Razvi model underestimates the influence of concrete on the confinement effect to increase the strength. Fig. 10 shows the predicted and measured strain at maximum stress. Generally speaking, the modified Kent-Park model underestimates, and the Mander model overestimates. The Sheikh-Uzumeri and the Saatcioglu-Razvi model predicts the actual measured strain at maximum stress reasonably well. However, all models overestimates or underestimates ε_{c50c} as seen in Fig. 11, indicating that all existing models failed to predict the behavior of confined concrete.

This is explained by the fact that most of these models were proposed based on the result of limited in scope. That is, these models are based on the results of rather limited experiments on small scale test specimens and did not fully consider a variety of influential variables. It has been pointed out that the stress of transverse steel at maximum stress (strength) (f_s) was used as the yield strength for the transverse steel to overestimate the transverse confinement pressure. Additionally, important variables were not properly and effectively evaluated.

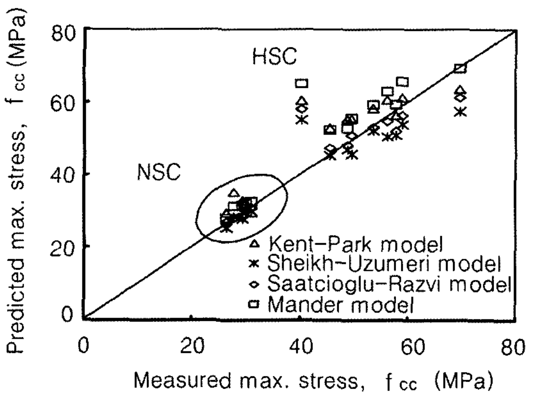


Fig. 9 Comparison of previous model with test results(f_{cc})

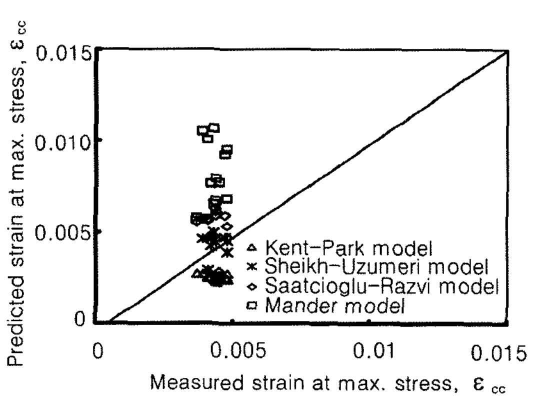


Fig. 10 Comparison of previous model with test results(ε_{cc})

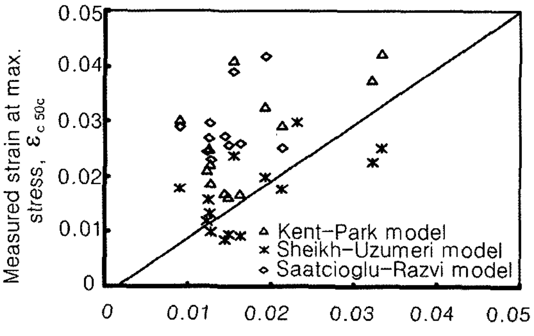


Fig. 11 Comparison of previous model with test results(ε_{c50c})

Thus, an assertion is made hereby to require the development of a rational and practical model, that can predict the behavior of high strength confined concrete accurately, and for it to be reflected on the design of such a model in serious need.

6. Conclusions

The following conclusions are derived from the experimental results of this study.

1) Although high strength concrete columns is expected to possess higher strength and improved ductility by transverse steel, a more effective confinement is in need to achieve the increase in strength and ductility in the same proportion to lower strength concrete. In other words, 50% more of transverse reinforcing steels was required to increase the concrete strength from 250 kgf/cm^2 to about 500 kgf/cm^2.

2) The increase in volumetric ratio can directly improve both the strength and ductility of confined concrete, and it is deemed that the more effective configuration type of transverse steel (E and F-type) can result in the increase of the concrete strength. However, since the volumetric ratio of transverse steel does not increase in linear proportion to the increase in strength, the precise delineation of this relationship is important in determining or proposing the best confinement model.

3) Given the same volumetric ratio of transverse steel, a better confinement effect can be expected with the more effective configuration type of the transverse steel. Thus, it follows that a method to quantify the configuration of transverse steel as an important variable of confinement effect is in need.

4) Although existing models predicts the maximum stress of normal strength concrete fairly well, the modified Kent-Park model and Mander model overestimate it by 14% and 15%, respectively. Moreover, Sheikh-Uzumeri and Saatcioglu-Razvi models overestimate the maximum stress as much as 40%. Modified Kent-Park model and Sheikh-Uzumeri model underestimate the strain at maximum stress by about 45% while Mander and Saatcioglu-Razvi models overestimate it by about 50%. Thus, it was found that most models failed to predict the descending part of the stress-strain relationship reasonably.

5) It has been known that confining the concrete with transverse reinforcing steels significantly influences the behavior of concrete columns including the increase in the strength and ductility of the member. However, it is also the current situation that there is hardly any confinement model for concrete to predict accurately the behavior of high strength concrete. Thus, it can be safely asserted that the development of a rational and practical model to predict the behavior of confined high-strength concrete and to reflect the prediction on the analysis of the behavior of confined high-strength concrete is not only in need but required.

Acknowledgements

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