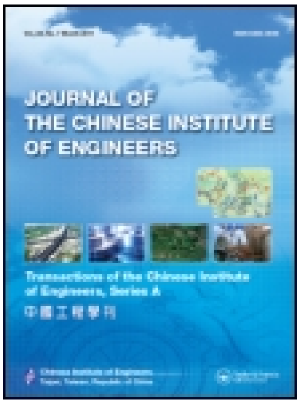


Journal of the Chinese Institute of Engineers
Publication details, including instructions for authors and subscription information:
http://www.tandfonline.com/loi/tcie20

A relative rigidity approach for design of concrete-encased composite columns

Cheng Chiang Weng [a], Sheng I. Yen [b] & Huei Shun Wang [b]
[a] Department of Civil Engineering, National Chiao Tung University, Hsinchu, Taiwan 300, R.O.C. E-mail:
[b] Department of Civil Engineering, National Chiao Tung University, Hsinchu, Taiwan 300, R.O.C.
Published online: 04 Mar 2011.

A RELATIVE RIGIDITY APPROACH FOR DESIGN OF CONCRETE-ENCASED COMPOSITE COLUMNS

Cheng Chiang Weng*, Sheng I Yen, and Huei Shun Wang

ABSTRACT

Presented herein is a new design approach utilizing the concept of relative rigidity for the design of concrete-encased composite columns. The new approach takes into account the relative rigidity ratio (RRR) of the steel portion and the reinforced concrete (RC) portion in a composite column to calculate the axial loads shared by each of the participating materials. The proposed approach also adopts the concept of strength superposition to sum up the calculated flexural strengths of the steel and the RC portions in a composite column. By utilizing the concepts of "relative rigidity" and "strength superposition", it becomes feasible to combine the column design equations in the ACI-318 Code and the AISC-LRFD Specification to create a new design approach for composite columns. To evaluate the accuracy of this method, the column strengths predicted by the proposed approach are compared to the results calculated using a numerical fiber analysis and to the results of 28 composite columns tested by previous researchers. The comparisons show that the proposed new approach gives satisfactory predictions of the strengths of the composite columns.

Key Words: concrete-encased composite column, relative rigidity, strength superposition, column test results, fiber analysis, ACI-318 code, AISC-LRFD specification.

I. INTRODUCTION

The concrete-steel composite structural system produces a building with advantages which include the stiffness of reinforced concrete and the strength of structural steel. Some additional merits for concrete-encased composite structural members are that the concrete also protects the steel section from fire damage and local buckling failure. In the United States, the design provisions for composite columns can be found in the ACI (American Concrete Institute) Code or in the AISC (American Institute of Steel Construction) Specification. The ACI-318 Code (2005) was the sole major reference for composite column design until the first publication of the AISC-LRFD (Load and Resistance Factor Design) Specification in 1986. As far as the design methodologies for composite columns are concerned, in section I2 of the AISC-LRFD Specification (2005), the design of a composite column is performed first by transforming the reinforced concrete portion into an equivalent amount of steel section. Then, the composite column is designed using the equations developed for steel columns. On the contrary, the ACI-318 Code considers the steel section as an equivalent amount of reinforcement so that the composite column is designed as an ordinary reinforced concrete column.

Considering the existing design methods in the American building codes, it is the writers' observation that the direct application of the design equations originally developed for reinforced concrete columns or for steel columns may not be appropriate for the design of concrete-encased composite columns. For instance, the nominal strength of a composite column specified in the ACI-318 Code is based on the assumption of strain compatibility; however, the influence of residual stress in the steel section is neglected. On the other hand, it is noted that the AISC-LRFD column equations are significantly influenced by the residual stress and the initial-out-of-straightness of the steel column (Bjohovde and Tall, 1971; SSRC, 1988); however, the residual stress and the initial

*Corresponding author. (Email: frankweng@mail.nctu.edu.tw)
The authors are with the Department of Civil Engineering, National Chiao Tung University, Hsinchu, Taiwan 300, R.O.C.

imperfection play a much less significant role in the concrete-encased composite columns.

In Japan, a type of composite construction called "steel reinforced concrete (SRC)" has long been popular since the second world war (Wakabayashi *et al*., 1971). The concept of "simple superposition method (SSM)" for composite column design was adopted in the AIJ (Architecture Institute of Japan) SRC Code (2001). The strength of a composite column can be easily determined through superposition of the strengths of the steel and the RC portions of the composite column. However, the bond effect between the steel and the concrete is conservatively neglected in the SSM. Although the concept of SSM is simple and straightforward, however, for engineers who are not familiar with the Japanese building codes, the use of AIJ-SRC Code could become a difficult task.

It is observed that by adopting the concept of strength superposition, we may be able to combine the widely used AISC-LRFD Specification and the ACI-318 Code to create an alternative new approach for the design of concrete-encased composite columns. Recent studies by Weng *et al*. (2001, 2002) have successfully adopted this approach in predicting the shear strength of concrete-encased composite structural members. In this study, the proposed new approach also takes into account the relative rigidity ratio (RRR) of the steel portion and the RC portion in a composite column to calculate the external loads shared by each of the participating materials. To evaluate the accuracy of this method, the column strengths predicted by the proposed approach are compared to the results calculated using a numerical fiber analysis and to the results of 28 composite columns tested by previous researchers.

II. REVIEW OF EXISTING DESIGN APPROACHES

In the United States, design provisions for composite columns are included in two different sets of structural design codes. One is the ACI-318 Code and the other is the AISC-LRFD Specification. Both the ACI and the AISC provisions are applicable to concrete-encased steel columns and concrete-filled tubular (CFT) columns (BSSC, 1997). It is noted that the above-mentioned specifications often give significantly different values of calculated member strengths due to the difference in the design methodologies (Furlong, 1983; El-Tawil, 1995; Viest *et al*., 1997).

1. ACI-318 Approach

The following paragraphs briefly review the concerned strength provisions for concrete-encased composite columns as recommended in section 10.16 of the ACI-318 Code (2005).

(i) Axial Compressive Strength

The nominal axial compressive strength of a concrete-encased composite short column, P_0, can be found by summing up the axial capacities of the materials that make up the cross section. That is

$$P_0 = 0.85 f_c' A_c + F_{yr} A_r + F_y A_s \tag{1}$$

where P_0 = column capacity under axial compression; f_c' = specified compressive strength of concrete; A_c = area of concrete; F_{yr} = specified yield strength of longitudinal reinforcement; A_r = area of longitudinal reinforcement; F_y = specified yield strength of steel section; A_s = area of steel section. For tied columns, the maximum axial compressive strength P_n is limited to $0.8P_0$ owing to a minimum eccentricity which is assumed under the axial load.

(ii) Flexural and Axial Loads

The ACI-318 design provisions for the strength interaction between the axial and the flexural loads for concrete-encased composite columns are essentially the same as those for ordinary reinforced concrete columns. They are based on a strain compatibility analysis at the ultimate state to develop a thrust-moment (P-M) interaction relationship. The following assumptions are made in the analysis:

1. Plane section remains plane;
2. The maximum concrete compressive strain is limited to 0.003;
3. The Whitney stress block is used for the concrete strength calculation;
4. Tensile strength of the concrete is neglected;
5. Strain hardening of steel section and rebar is neglected.

2. AISC-LRFD Approach

Although the AISC Specification has included design provisions for composite beams with shear connectors since 1961, design requirements for composite columns were not recommended until the publication of the first edition of the AISC-LRFD Specification in 1986. The concept of extending steel column design methodology to the composite columns using modified properties was first introduced by Furlong (1976). Modified yield stress F_{my}, modulus of elasticity E_m and radius of gyration r_m were incorporated into steel column design equations for the design of composite columns. This procedure was presented by Task Group 20 of the Structural Stability Research Council (SSRC) in 1979. The following paragraphs briefly review the concerned strength provisions for concrete-encased composite columns as recommended in Chapter I of the AISC-LRFD Specification (2005).

(i) Axial Compressive Strength

The axial compressive strength of a concrete-encased composite column can be determined by using the same equations as for bare steel columns except that the formulas are being entered with modified properties F_{my}, E_m and r_m. The nominal axial compressive strength of a composite column is given as

$$P_n = A_s F_{cr}, \tag{2}$$

where A_s is the area of the steel section, and F_{cr} is the critical stress of the column given by the following equations:

$$F_{cr} = (0.658^{\lambda_c^2})F_{my} \quad \text{for } \lambda_c \le 1.5 \tag{3}$$

and

$$F_{cr} = (\frac{0.877}{\lambda_c^2})F_{my}, \quad \text{for } \lambda_c > 1.5 \tag{4}$$

where λ_c = the slenderness parameter; F_{my} = modified yield stress.

(ii) Flexural and Axial Loads

For a composite column symmetrical about the plane of bending, the interaction of the compressive and the flexural loads should be limited by the following bilinear relationship:

$$\frac{P_u}{\phi_c P_n} + \frac{8M_u}{9\phi_b M_n} \le 1.0 \quad \text{for } P_u \ge 0.2\phi_c P_n \tag{5}$$

and

$$\frac{P_u}{2\phi_c P_n} + \frac{M_u}{\phi_b M_n} \le 1.0, \quad \text{for } P_u < 0.2\phi_c P_n \tag{6}$$

where P_u = factored axial load; M_u = factored moment; P_n = nominal axial compressive capacity; M_n = nominal flexural capacity without axial force; ϕ_c = resistance factor for compression, taken as 0.85; ϕ_b = resistance factor for bending, taken as 0.9.

Based on the assumption of plastic stress distribution on the composite section, the commentary of section I4 of the AISC-LRFD Specification provides an equation to determine the pure bending capacity for a doubly symmetric composite section. That is

$$M_n = M_p = ZF_y + \frac{1}{3}(h_2 - 2c_r)A_r F_{yr} + (\frac{h_2}{2} - \frac{A_w F_y}{1.7 f_c' h_1})A_w F_y, \tag{7}$$

where Z = plastic section modulus of steel section; h_2 = concrete thickness in the plane of bending; c_r = thickness of concrete cover from center of rebar to the edge of section in the plane of bending; h_1 = concrete width perpendicular to the plane of bending; A_w = web area of steel section.

III. THE PROPOSED NEW DESIGN APPROACH

1. Methodology of the Proposed Approach

For a concrete-encased composite column subjected to combined axial load and bending moment, the proposed new design approach firstly makes use of the "relative rigidity ratio (RRR)" of the steel portion and the reinforced concrete (RC) portion in the composite column to calculate the axial loads shared by each of the participating materials, P_s and P_{rc}. Secondly, after finding the axial loads shared by the steel and the RC portions, the corresponding flexural capacities, M_s and M_{rc}, of each of the participating materials are calculated directly by using the existing P-M interaction formulas given in the AISC-LRFD Specification (2005) and the ACI-318 Code (2005), respectively. Finally, the proposed approach adopts the concept of "strength superposition" to sum up the flexural capacities, M_s and M_{rc}, which gives the total flexural strength of the composite column, M_{comp}.

Since a concrete-encased composite column is physically the combination of a steel column and a reinforced concrete column, it is noted that by means of strength superposition, it becomes feasible to combine the widely used column design formulas suggested in the AISC Specification and the ACI Code to develop an alternative and straightforward new approach for the design of composite columns.

Figure 1 illustrates a conceptual flow chart of the proposed new approach for the design of composite columns. More detailed flow charts are presented in Figs. 2 and 3.

In general, for a composite column subjected to combined axial load and bending moment, the proposed new design approach includes the following major steps:

(1) Calculate the axial loads shared by the steel portion and by the RC portion, P_s and P_{rc}, respectively, according to their relative rigidity ratios.

(2) Having the axial loads P_s and P_{rc}, calculate the corresponding flexural capacities of the steel portion and the RC portion, M_s and M_{rc}, by using the *P-M* interaction formulas given in the AISC-LRFD Specification and the ACI-318 Code, respectively.

(3) Superpose the flexural capacities of the steel portion and the RC portion to obtain the total

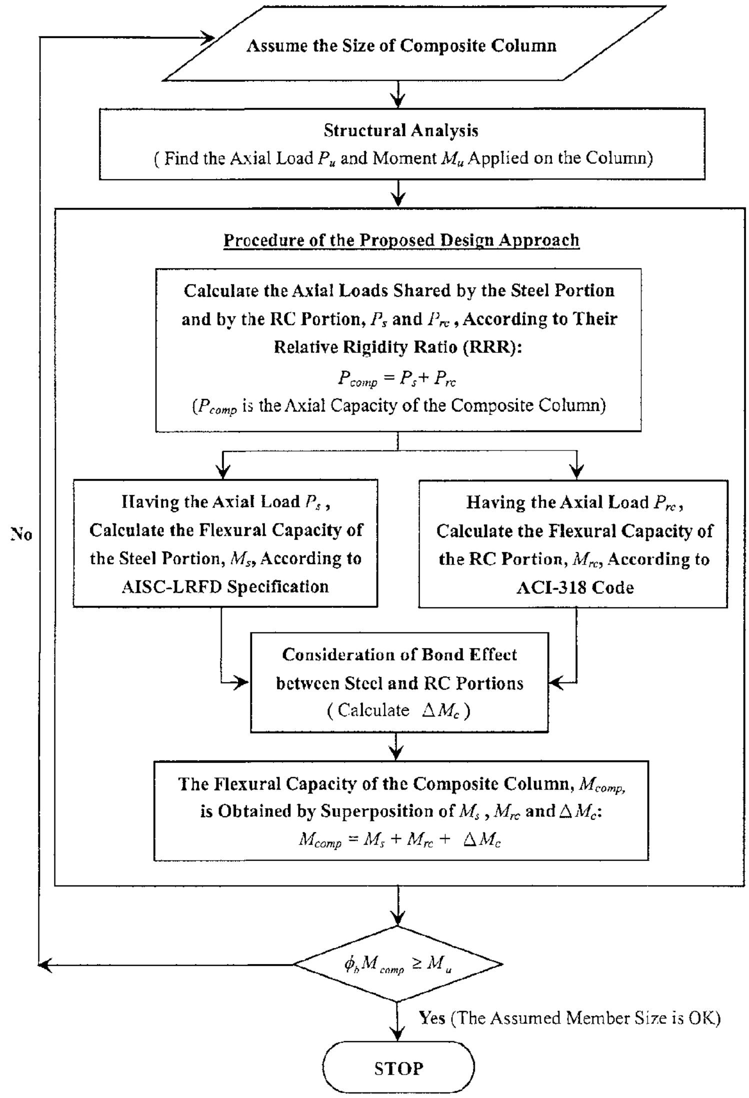


Fig. 1 Conceptual Flow Chart of the Proposed New Approach for Composite Column Design

flexural capacity of the composite column, M_{comp}.

(4) Check if the design of the composite column is satisfactory. The design is satisfactory if the design flexural strength of the composite column, $\phi_b M_{comp}$, is larger than the required external bending moment, M_u, where ϕ_b is the flexural strength reduction factor. That is

$$\phi_b M_{comp} \geq M_u. \tag{8}$$

It is noted that in Section 9.3.2.1 of the current ACI 318-05 Code, ϕ_b is taken as 0.9 for tension controlled section ($\varepsilon_t \geq 0.005$) only. If ε_t is less than 0.005, the section becomes a transition section and ϕ_b needs to be determined by linear interpolation between 0.65 and 0.9. A lower ϕ_b factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

2. Load Sharing According to Relative Rigidity Ratio

As shown in Fig. 2, the proposed design approach utilizes the concept of relative rigidity to calculate the axial loads shared by the steel portion and by the RC portion of the composite column. The criterion of $KL \leq 12D$ used in the AIJ-SRC Code (2001) is adopted in this study to represent a stocky composite column, and $KL > 12D$ for a slender composite column; where KL is the effective length of the composite column; and D is the overall dimension of the composite column in the direction of bending.

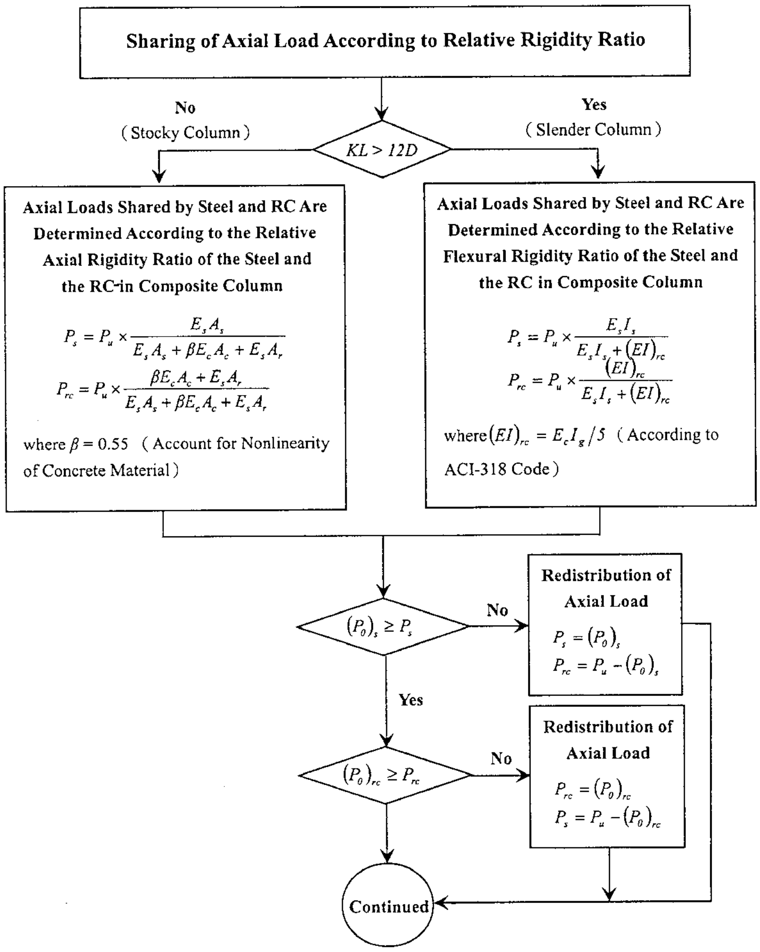


Fig. 2 Flow Chart for Calculation of Axial Loads Shared by the Steel Portion and by the RC Portion of a Composite Column According to Relative Rigidity Ratio

For a short composite column ($KL \leq 12D$), the axial rigidity, EA, is used to calculate the axial loads shared by the steel portion and by the RC portion. This gives

$$P_s = P_u \times \frac{E_s A_s}{E_s A_s + \beta E_c A_c + E_s A_r} \tag{9}$$

$$P_{rc} = P_u \times \frac{\beta E_c A_c + E_s A_r}{E_s A_s + \beta E_c A_c + E_s A_r} . \tag{10}$$

For a slender composite column ($KL > 12D$), the flexural rigidity, EI, is used to calculate the axial loads shared by the steel portion and by the RC portion. This gives

$$P_s = P_u \times \frac{E_s I_s}{E_s I_s + (EI)_{rc}} , \tag{11}$$

$$P_{rc} = P_u \times \frac{(EI)_{rc}}{E_s I_s + (EI)_{rc}} , \tag{12}$$

where P_u = applied axial load; E_s = elastic modulus of steel; E_c = elastic modulus of concrete; A_s = area of steel portion; A_c = area of concrete portion; A_r = area of longitudinal reinforcement; β = reduction coefficient; I_s = moment of inertia of steel portion; $(EI)_{rc}$ = flexural rigidity of RC portion. $(EI)_{rc}$ is conservatively taken as $(E_c I_g)/5$ according to the ACI-318 Code.

The reduction coefficient β in Eqs. (9) and (10) is to account for the nonlinear behavior of concrete under axial compression. It is known that the normal weight concrete behaves nearly linearly only up to about $0.5 f_c'$. The elastic relationship can not be hold beyond this stress level. Thus a reduction coefficient β is introduced to account for the nonlinear behavior of concrete under large axial load. As shown in Fig. 4, the secant modulus of the concrete, $(E_c)_{sec}$, can be found from

$$(E_c)_{\text{sec}} = \frac{f_c'}{\varepsilon_{peak}} . \tag{13}$$

Let the β be the ratio of $(E_c)_{sec}$ to E_c, that is

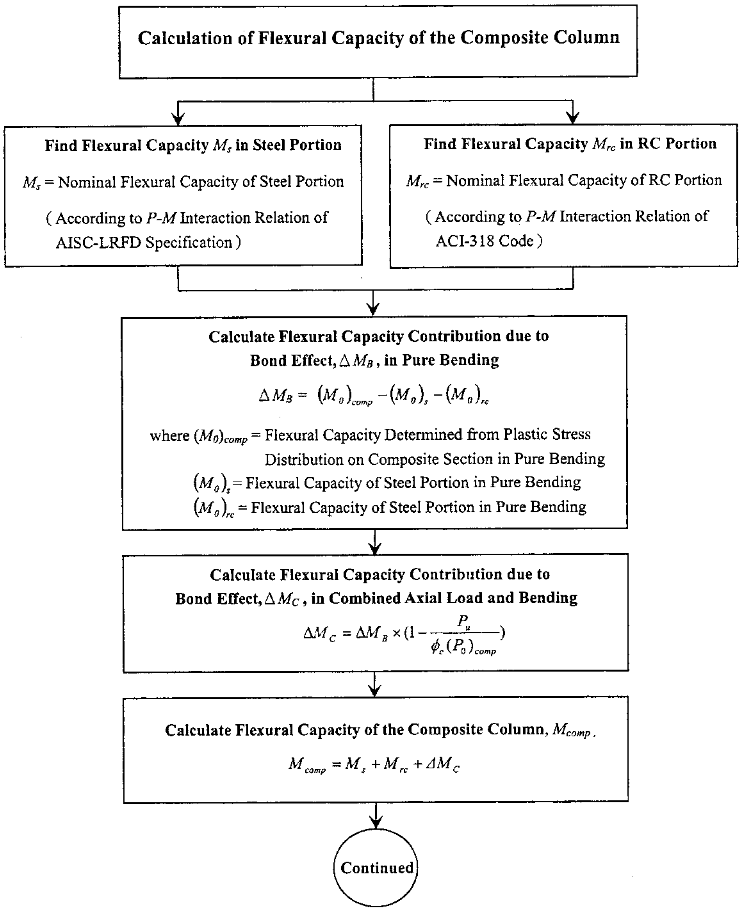


Fig. 3 Flow Chart for Calculation of Flexural Capacity of a Composite Column

$$\beta = \frac{(E_c)_{\sec}}{E_c}. \tag{14}$$

The strains ε_{peak} corresponding to the peak stresses f_c' can be found in the normal weight concrete stress-strain curves given in Nilson *et al.* (2003) and the results are listed in Table 1. In this study, an average β value of 0.55 is adopted to approximately account for the non-linear behavior of concrete under large axial load.

In order to achieve a more economical design, this study suggests that if the external axial load shared by each of the steel or the RC portion, P_s or P_{rc}, exceeds the specified strength supported by itself, a redistribution of the axial load can be performed. As shown in the lower part of Fig. 2, if the axial load shared by the steel portion, P_s, calculated from Eq. (9) or (11), exceeds the nominal axial strength of itself, $(P_0)_s$, then the RC portion is allowed to share more load through the redistribution of the axial load. This gives

$$\left.\begin{aligned} P_s &= (P_0)_s \\ P_{rc} &= P_u - (P_0)_s \end{aligned}\right\} \tag{15}$$

Similarly, if the axial load shared by the RC portion, P_{rc}, which calculated from Eqs. (10) or (12), is larger than the nominal axial strength of itself, $(P_0)_{rc}$, then the steel portion is allowed to share more load through the redistribution of the axial load. This gives

$$\left.\begin{aligned} P_{rc} &= (P_0)_{rc} \\ P_s &= P_u - (P_0)_{rc} \end{aligned}\right\} \tag{16}$$

3. Calculation of the Flexural Capacity

For a composite column subjected to combined axial load and bending moment, the proposed design procedure shown in Fig. 3 suggests that, after finding the axial loads P_s and P_{rc} shared by the steel and the RC portions, the corresponding flexural capacities, M_c and M_{rc}, of each of the participating materials can be calculated directly by using the *P-M* interaction formulae given in the current AISC-LRFD Specification and the ACI-318 Code, respectively. Finally, the proposed approach adopts the concept of "strength superposition" to sum up the flexural capacities, M_s and M_{rc}, which gives the total flexural

Table 1 The Reduction Coefficient β of Elastic Modulus of Normal Weight Concrete at Different Specified Compressive Strengths

f_c' (MPa)	$E_c^{(1)}$ (MPa)	$\varepsilon_{peak}^{(2)}$	$(E_c)_{sec} = f_c'/\varepsilon_{peak}$ (MPa)	$\beta = (E_c)_{sec}/E_c$
20.7	21384	0.00196	10553	0.493
27.6	24692	0.00200	13790	0.558
34.5	27606	0.00227	15187	0.550
41.4	30131	0.00244	16954	0.563
55.2	34919	0.00262	21053	0.603
			Average value of β :	0.553

Note: (1): $E_c = 4700\sqrt{f_c'}$, as suggested by ACI-318 Code.
(2): ε_{peak} = strain of concrete when its compressive strength reaches f_c'.

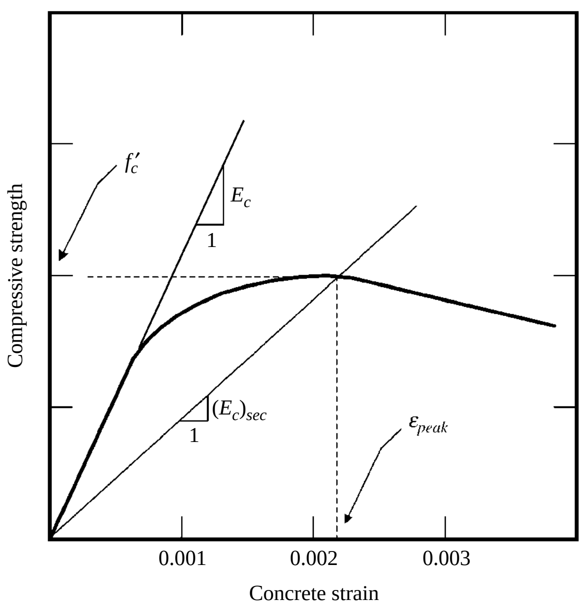


Fig. 4 Reduction of Elastic Modulus of Concrete Obtained by Using Tangent Modulus Approach (E_c) and by Secant Modulus Approach $(E_c)_{sec}$

strength of the composite column, M_{comp}.

It is noted that for a concrete-encased composite structural member subjected to pure bending, the contribution of flexural capacity due to the bond effect between the steel section and the concrete, ΔM_B, can be expressed as

$$\Delta M_B = (M_0)_{comp} - (M_0)_s - (M_0)_{rc}, \tag{17}$$

where $(M_0)_{comp}$ = flexural capacity including bond effect of the composite structural member in pure bending; $(M_0)_s$ = flexural capacity of the steel portion in pure bending; $(M_0)_{rc}$ = flexural capacity of the RC portion in pure bending.

In Eq. (17), the calculation of $(M_0)_{comp}$ is often complicated due to its composite nature. Nevertheless, it is noted that in the commentary to section I4 of the AISC-LRFD Specification, an equation based on the plastic stress distribution in the composite section is suggested for calculating the value of $(M_0)_{comp}$. In order to simplify the design procedure, this equation (as cited in Eq. 7 of this paper) is adopted in the current study to calculate the flexural capacity of the composite structural member subjected to pure bending.

IV. VERIFICATION ANALYSIS

To evaluate the performance of the proposed design approach, this study compares the composite column strengths predicted by the proposed approach to those calculated using a numerical fiber analysis and to 28 composite column test results reported by previous researchers. All of the column specimens were subjected to combined axial load and bending moment.

1. Numerical Fiber Analysis

The fiber analysis has been used successfully in predicting the strengths of composite sections by many researchers (El-Twain *et al.*, 1995 and 1999; Munzo and Hsu, 1997). In this study, a general purpose computer program, BIAX, developed by Wallace (1989) of University of California at Berkeley is used to evaluate the strength of a composite column. As shown in Fig. 5, a composite column cross-section is discretized into small fibers. The column strength is determined based on the following equations:

$$P = \sum_{i=1}^{nc} f_i A_i + \sum_{j=1}^{ns} f_j A_j + \sum_{k=1}^{nb} f_k A_k \tag{18}$$

$$M = \sum_{i=1}^{nc} f_i A_i y_i + \sum_{j=1}^{ns} f_j A_j y_j + \sum_{k=1}^{nb} f_k A_k y_k , \tag{19}$$

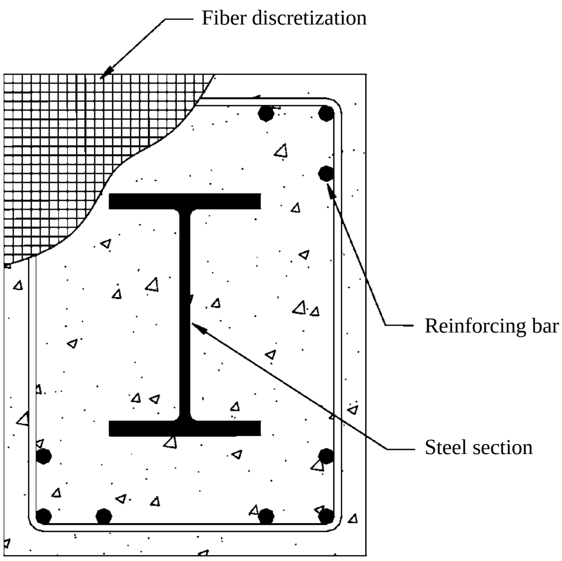
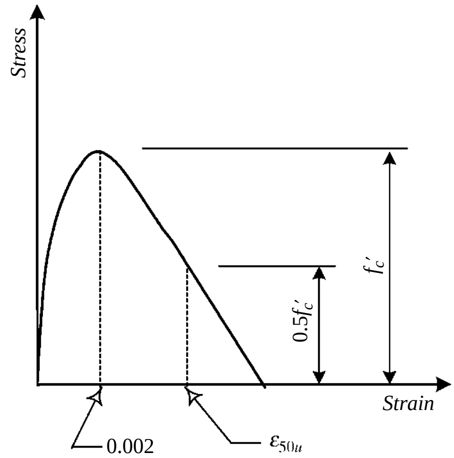
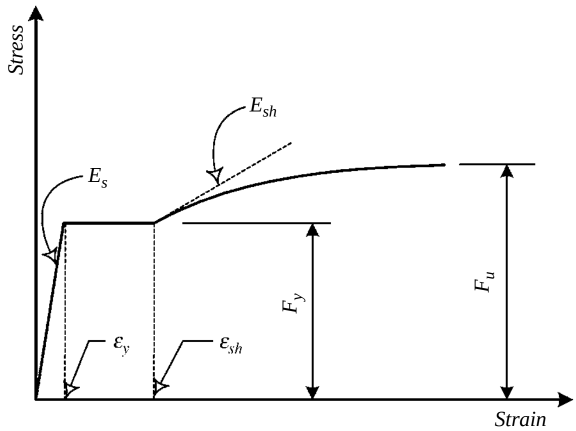


Fig. 5 Discretized Composite Column Cross-section for Numerical Fiber Analysis



(a) Concrete stress-strain relationship (Kent and Park, 1971)



(b) Steel stress-strain relationship (Vecchio and Collins, 1986)

Fig. 6 Material Constitutive Models: (a) Concrete; (b) Steel

where P = ultimate compressive strength; M = ultimate bending strength; f_i, f_j, f_k = effective stress at centroid of concrete fiber i, steel section fiber j and rebar k, respectively; A_i, A_j, A_k = area of concrete fiber i, steel section fiber j and rebar k, respectively; y = centroid coordinates of fiber i or j or k; nc, ns, nb = number of concrete fibers, steel section fibers and rebars, respectively.

The stresses in Eqs. (18) and (19) can be calculated using the fiber strains and constitutive relations where the strain is a function of the neutral axis and the extreme compressed concrete fiber. In this study, the maximum strain of extreme compressed concrete fiber is taken as 0.003 for fiber analysis. For the stress-strain relationship as shown in Fig. 6, this study follows the models proposed by Kent and Park (1971) and by Vecchio and Collins (1986) for concrete and steel, respectively.

2. Previous Tested Composite Columns

The test results of 28 concrete-encased composite columns were collected and summarized in Table 2. All column specimens were subjected to combined axial load and bending moment. In this table, the specimens are divided into four groups, the M, R, N and W-groups, representing the specimens were tested by Mirza (1996), Ricles (1994), Naka (1979) and Wakabayashi (1971), respectively. A brief review of each group is given as follows.

In 1996, Mirza et al. studied fourteen encased composite columns subjected to strong axis bending. As observed from the tests, concrete strain in extreme compression fiber reached around 0.0025 to 0.004 prior to failure of specimens. In addition, the test results indicated that the bonding at the interface of the steel flange and the surrounding concrete had little effect on the ultimate strength of the composite columns.

Ricles *et al.* (1994) presented experimental results from eight encased composite columns subjected to strong axis bending. Each of the columns was tested under monotonic axial load and cyclically applied lateral load. It was observed that the maximum capacity of the specimens developed after yielding of the longitudinal reinforcements and the steel flange. The test results also showed that the shear studs were not effective in enhancing the flexural strength of the composite columns.

Naka *et al.* (1977) carried out tests on three pinned-end composite columns. All specimens were subjected to strong axis bending, and the applied loading included combinations of axial and bending forces. It was reported that the failure mode of the specimens could be divided into two categories. One

Table 2 Composite Column Test Data from Previous Researchers

Specimen	B × D (mm)	Steel section $d_s \times b_f \times t_w \times t_f$ (mm)	A_r (mm^2)	F_{ys} (MPa)	F_{yr} (MPa)	f_c' (MPa)	M_{test} (kN-m)	P_{test} (kN)
Test data from Mirza (1996)								
M1	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	27.0	64.1	950.0
M2	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	27.0	63.2	550.0
M3	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	27.6	78.2	570.0
M4	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	24.8	66.0	154.3
M5	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	28.5	65.6	95.0
M6	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	27.4	82.2	925.0
M7	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	27.4	76.0	775.0
M8	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	26.5	82.3	540.0
M9	240 × 240	96 × 100 × 5.1 × 8.6	284	293.4	565.0	27.2	73.5	107.5
M10	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	27.4	72.0	927.0
M11	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	27.4	69.9	720.0
M12	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	25.5	83.0	540.0
M13	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	25.5	79.9	296.0
M14	240 × 240	96 × 100 × 5.1 × 8.6	284	311.2	634.0	25.5	68.7	100.0
Test data from Ricles (1994)								
R1	406 × 406	W8 × 40	3148	373.7	455.8	32.7	626.0	1490.0
R2	406 × 406	W8 × 40	1548	373.7	434.4	34.5	593.0	1490.0
R3	406 × 406	W8 × 40	4645	373.7	434.4	30.9	784.0	1490.0
R4	406 × 406	W8 × 40	2581	373.7	448.2	31.1	670.0	1490.0
R5	406 × 406	W8 × 40	4645	373.7	434.4	34.5	776.0	1490.0
R6	406 × 406	W8 × 40	2581	373.7	448.2	35.8	667.0	1490.0
R7	406 × 406	W8 × 40	4645	373.7	434.4	62.9	840.0	1490.0
R8	406 × 406	W8 × 40	4645	373.7	434.4	64.5	832.0	1490.0
Test data from Naka (1977)								
N1	240 × 300	180 × 120 × 4.5 × 12	2323	344.8	461.3	25.5	197.4	1470.0
N2	240 × 300	180 × 120 × 4.5 × 12	2323	344.8	461.3	25.5	235.0	980.0
N3	240 × 300	180 × 120 × 4.5 × 12	2323	344.8	461.3	25.5	228.4	490.0
Test data from Wakabayashi (1971)								
W1	210 × 210	150 × 100 × 6 × 9	284	306.1	360.6	26.4	72.4	293.6
W2	210 × 210	150 × 100 × 6 × 9	284	306.1	360.6	28.9	67.7	587.1
W3	210 × 210	150 × 100 × 6 × 9	284	306.1	360.6	27.0	59.0	880.7

was concrete crushing failure and local buckling of steel flange due to compression; the other was concrete crushing failure, and buckling of rebar due to compression and yielding of rebars due to tension.

Wakabayashi *et al*. (1971) presented experimental results from three concrete-encased composite columns subjected to strong axis bending. The applied loading included static axial load and transverse force. It was observed that as the load was increased to the failure condition, the concrete outside rebars spalled, rebars on the compression side buckled, and rebars on the tension side yielded for most of the tested specimens.

3. Comparative Results between Proposed Approach and Fiber Analysis

In Table 3, the composite column test results of previous researchers are compared with the predicted capacities using the proposed approach and numerical fiber analysis. The column strengths calculated according to the ACI-318 Code (2005) and the AISC-LRFD Specification (2005) are also shown in the table. In this table, M_{test} is the ultimate moment capacity observed from the test; M_{prop} is the predicted moment strength using the proposed method; and M_{fiber} is the predicted moment strength using the fiber analysis.

The ratios listed in column (9) of Table 3 are the predicted strength-to-tested strength ratios of the moment capacities, M_{prop}/M_{fiber}. As shown in the table, the maximum and minimum ratios of M_{prop}/M_{fiber}, are 1.14 and 0.85, respectively. In addition, the average value of the strength ratio is 0.99, with coefficient of variation (COV) of 10%.

Table 3 Comparison of Composite Column Test Results and Strengths Predicted by the Proposed Method, ACI-318 Code, AISC-LRFD Specification and Fiber Analysis

Specimen	M_{test} (kN-m)	M_{ACI} (kN-m)	M_{LRFD} (kN-m)	M_{fiber} (kN-m)	M_{prop} (kN-m)	Predicted strength ratio $\frac{M_{ACI}}{M_{test}}$	$\frac{M_{LRFD}}{M_{test}}$	$\frac{M_{prop}}{M_{fiber}}$	$\frac{M_{prop}}{M_{test}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
M1	64.1	53.0	41.7	60.8	69.0	0.83	0.65	1.14	1.08
M2	63.2	68.8	53.8	64.7	66.0	1.09	0.85	1.02	1.05
M3	78.2	61.1	47.3	66.2	59.7	0.78	0.60	0.90	0.76
M4	66.0	57.4	49.5	61.8	52.8	0.87	0.75	0.85	0.80
M5	65.6	56.3	48.0	64.2	55.5	0.86	0.73	0.87	0.85
M6	82.2	64.9	51.9	67.8	75.1	0.79	0.63	1.11	0.91
M7	76.0	65.9	52.6	68.7	75.9	0.87	0.69	1.11	1.00
M8	82.3	68.6	53.4	69.9	69.9	0.83	0.65	1.00	0.85
M9	73.5	58.5	49.5	65.1	57.6	0.80	0.67	0.89	0.78
M10	72.0	61.6	49.3	66.5	72.9	0.86	0.68	1.10	1.01
M11	69.9	64.3	51.4	68.6	75.8	0.92	0.73	1.11	1.08
M12	83.0	71.2	58.4	68.7	72.7	0.86	0.70	1.06	0.88
M13	79.9	69.5	58.5	70.7	66.2	0.87	0.73	0.94	0.83
M14	68.7	60.4	52.7	67.6	60.2	0.88	0.77	0.89	0.88
R1	626.0	579.6	467.2	622.7	617.2	0.93	0.75	0.99	0.99
R2	593.0	474.4	387.6	523.1	526.3	0.80	0.65	1.01	0.89
R3	784.0	612.5	529.7	700.1	649.4	0.78	0.68	0.93	0.83
R4	670.0	563.0	424.1	582.0	601.4	0.84	0.63	1.03	0.90
R5	776.0	630.9	527.9	722.3	663.1	0.81	0.68	0.92	0.86
R6	667.0	580.0	424.8	610.9	620.6	0.87	0.64	1.02	0.93
R7	840.0	705.9	591.5	829.2	722.0	0.84	0.70	0.87	0.86
R8	832.0	705.1	590.1	836.5	723.5	0.85	0.71	0.87	0.87
N1	197.4	184.4	140.4	198.1	211.7	0.93	0.71	1.12	1.07
N2	235.0	205.4	166.4	216.3	249.5	0.87	0.71	1.15	1.06
N3	228.4	220.6	187.8	233.1	251.8	0.97	0.82	1.08	1.10
W1	72.4	65.9	64.6	78.8	68.6	0.91	0.89	0.87	0.95
W2	67.7	65.6	54.8	74.6	71.3	0.97	0.81	0.96	1.05
W3	59.0	57.3	44.2	60.8	61.3	0.97	0.75	1.01	1.04
					Mean Value:	0.87	0.71	0.99	0.93
					Coefficient of Variation:	0.08	0.10	0.10	0.11

Evidences from the average ratio and the coefficient of variation show that the predicted strengths obtained by using the proposed approach are very close to those calculated by using the numerical fiber analysis.

4. Comparative Results between Proposed Approach and Test Results

Listed in column (10) of Table 3 are the predicted strength-to-tested strength ratios of the moment capacities, M_{prop}/M_{test}, for the 28 concrete-encased composite columns tested under combined axial load and bending moment. As shown in the table, the range of the predicted- to-tested ratios is from 0.76 to 1.10. Most of the strength ratios are between 0.85 and 1.05. The average value of the ratio is 0.93 with COV of 11%.

In addition to the comparison shown in Table 3, the P-M interaction diagrams calculated by using the proposed approach, the ACI-318 Code (2005) and the AISC-LRFD Specification (2005) are plotted in Figs. 7 to 10 for specimens of M, R, N and W groups, respectively. In these figures, the star symbol represents the tested strength of the composite columns.

For the M-group, the P-M interaction diagram based on the proposed approach is found to be conservative in most cases as compared to the test results. The average ratio of predicted strengths to test results is 0.91 with COV of 12%. For the R-group, the interaction diagram based on the proposed approach is also conservative as compared to the test results. The average ratio of predicted strengths to test results is 0.89 with COV of 6%.

Regarding the N-group, the P-M interaction

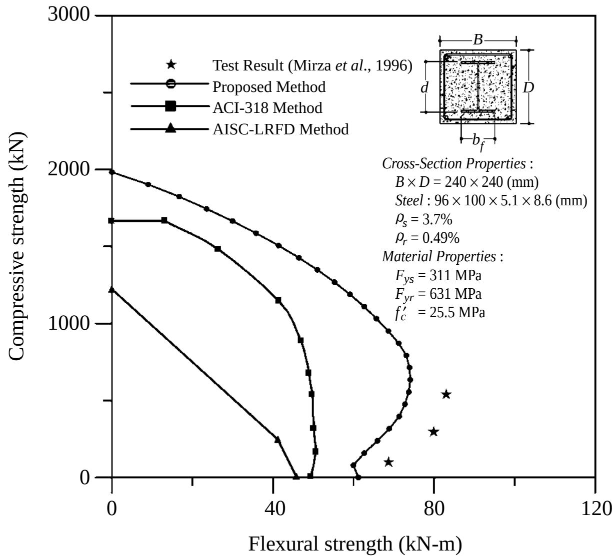


Fig. 7 Comparisons of Column Test Results and P-M Interaction Curves Predicted by Proposed Method, ACI-318 Method and AISC-LRFD Method: Specimens M12, M13 and M14

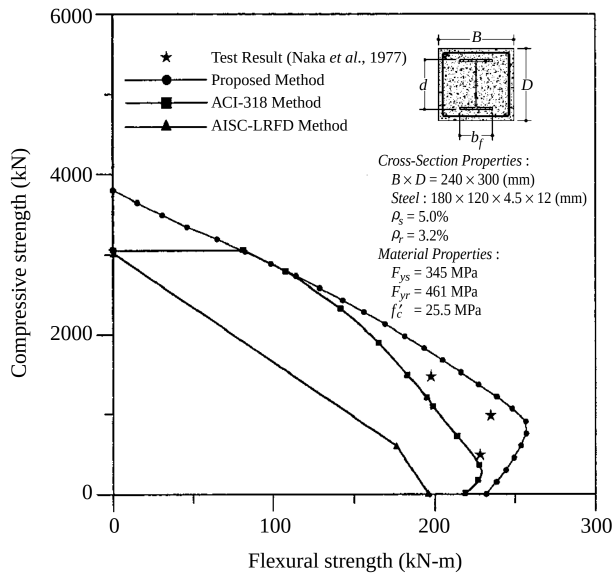


Fig. 9 Comparisons of Column Test Results and P-M Interaction Curves Predicted by Proposed Method, ACI-318 Method and AISC-LRFD Method: Specimens N1, N2 and N3

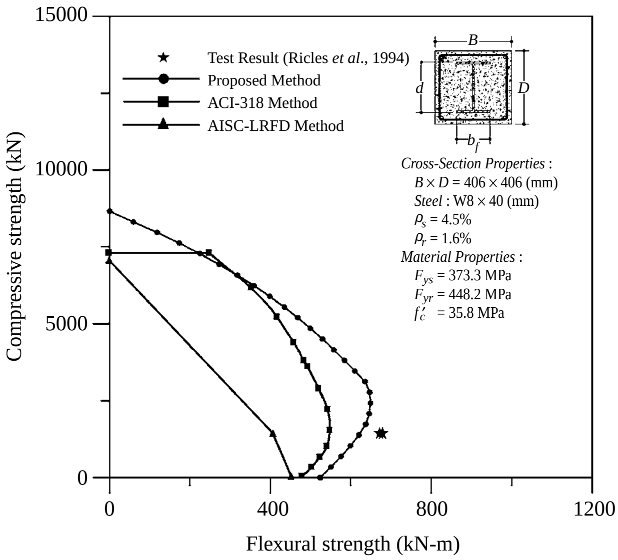


Fig. 8 Comparisons of Column Test Results and P-M Interaction Curves Predicted by Proposed Method, ACI-318 Method and AISC-LRFD Method: Specimens R4 and R6

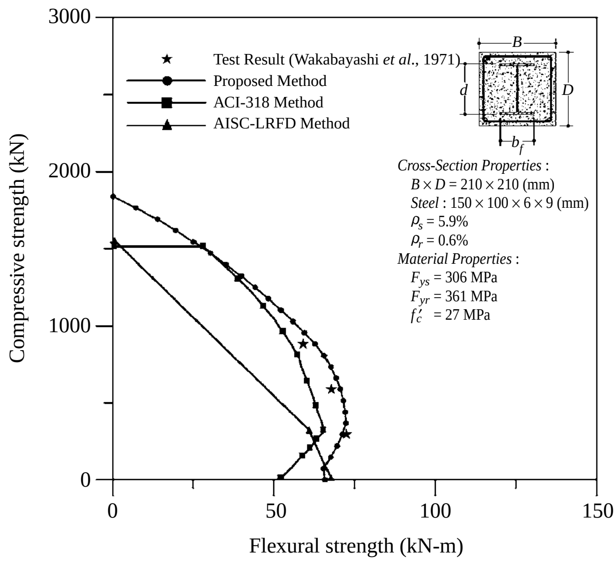


Fig. 10 Comparisons of Column Test Results and P-M Interaction Curves Predicted by Proposed Method, ACI-318 Method and AISC-LRFD Method: Specimens W1, W2 and W3

diagram based on the proposed method is slightly overestimated as compared to the test results. The average ratio of predicted strengths to test results is 1.08 with COV of 2%. The average of predicted strength ratios based on the proposed method to test results is 1.01 with COV of 6% for specimens of W-group.

The comparative results show that the proposed new approach gives satisfactory predictions of the strengths of concrete-encased composite columns. In general, this study has developed an alternative approach for the design of composite columns. The new design approach is based on the concepts of "relative rigidity" and "strength superposition" which are well-known concepts in structural mechanics and are familiar to most structural engineers.

V. CONCLUSIONS

The following conclusions can be drawn within the scope of this study:

(1) This study proposes an alternative design approach using the concept of "relative rigidity" for the design of concrete-encased composite columns. A new term called "relative rigidity ratio" is introduced in the proposed approach. This term represents the ratio of the rigidity of the steel portion to the reinforced concrete portion in the composite column. The external loads acting on a composite column are shared by each of the participating materials according to their relative rigidity ratios.

(2) In addition to the concept of relative rigidity, the proposed approach also adopts the concept of "strength superposition" to sum up the calculated strengths of the steel portion and the RC portion in the composite column. Since a composite column is physically the combination of a steel column and a concrete column, by strength superposition, it becomes feasible to combine the column design formulas used in the AISC-LRFD Specification and the ACI-318 Code to create an alternative new approach for the design of concrete-encased composite columns. Recent studies by Weng *et al.* (2001, 2002) have successfully adopted this method in predicting the shear strength of composite structural members.

(3) To evaluate the performance of the proposed approach, comparisons between the predicted strengths by the proposed approach and by a numerical fiber analysis are made. Finally, column test results reported by previous researchers were collected to evaluate the accuracy of the proposed approach. As compared to 28 column test results (all column specimens were subjected to combined axial load and bending moment) and to the values predicted by the fiber analysis, the proposed approach is proven to give satisfactory predictions of the strength of the concrete-encased composite columns.

ACKNOWLEDGMENTS

The financial support granted by the National Science Council of Taiwan R.O.C. is gratefully acknowledged.

REFERENCES

ACI, 2005, *Buildings Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*, American Concrete Institute, Farmington Hills, MI, USA.

AIJ, 2001, *Standards for Structural Calculation of Steel Reinforced Concrete Structures*, Architectural Institute of Japan, Tokyo, Japan.

AISC, 2005, *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, USA.

Bjohovde, R., and Tall, L., 1971, "Maximum Column Strength and Multiple Column Curve Concept," *Fritz Lab. Report No. 337.29*, Lehigh University.

BSSC, 1997, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures*, National Earthquake Hazards Reduction Program, Building Seismic Safety Council, Washington D.C., USA.

El-Tawil, S., and Deierlein, G. G., 1999, "Strength and Ductility of Concrete-Encased Composite Columns," *Journal of Structural Engineering*, ASCE, Vol. 125, No.9, pp. 1009-1019.

El-Tawil, S., Sanz-Picon, C. F., and Deierlein, G. G., 1995, "Evaluation of ACI 318 and AISC (LRFD) Strength Provisions for Composite Beam-Columns," *J. Construct. Steel Research*, Malta, Vol. 34, pp. 103-123.

Furlong, R. W., 1968, "Design of Steel-Encased Concrete Beam-Columns," *Journal of Structural Engineering*, ASCE, Vol. 94, No. 1, pp. 267-281.

Furlong, R. W., 1976, "AISC Columns Design Logic Makes Sense for Composite Column, Too," *Engineering Journal*, AISC, First Quarter, pp. 1-7.

Furlong, R. W., 1983, "Column Rules of ACI, SSLC and LRFD Compared," *Journal of Structural Engineering*, ASCE, Vol. 109, pp. 2375-2386.

Kent, D., and Park, R., 1971, "Flexural Members with Confined Concrete," *Journal of Structural Division*, ASCE, Vol. 97, pp. 1969-1990.

Mirza, S. A., Hyttinen, V., and Hyttinen, E., 1996, "Physical Tests and Analyses of Composite Steel-Concrete Beam-Columns," *Journal of Structural Engineering*, ASCE, Vol. 117, No. 8, pp. 2320-2339.

Muñoz, P. R., and Hsu, C.-T.T., 1997(a), "Behavior of Biaxial Loaded Concrete- Encased Composite Columns," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 9, pp. 1163-1171.

Muñoz, P. R., and Hsu, C.-T.T., 1997(b), "Biaxial Loaded Concrete-Encased Composite Columns: Design Equation," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 12, pp. 1576-1585.

Naka, T., Morita, K., and Tachibana, M., 1977, "Strength and Hysteretic Characteristics of Steel-Reinforced Concrete Columns (Part 2)," *Transaction of AIJ*, No. 250, Oct., pp. 47-58.

Nilson, A. H., Darwin, D., and Dolan, C. W., 2003, *Design of Concrete Structures*, 13th Ed., McGraw-Hill., New York, USA.

Ricles, J. M., and Paboojian, S. D., 1994, "Seismic Performance of Steel-Encased Composite Columns," *Journal of Structural Engineering*, ASCE, Vol. 120, No. 8, pp. 2474-2494.

SSRC, 1988, *Guide to Stability Design Criteria for*

Metal Structures, 4th Ed., Structural Stability Research Council (SSRC), John Wiley & Sons, New York, USA.

SSRC Task Group 20, 1979, "A Specification for the Design of Steel-Concrete Composite Columns," *Engineering Journal*, AISC, Vol. 4, pp. 101-115.

Vecchio, F., and Collins, M., 1986, "The Modified Compression Field Theory for Reinforced Concrete Subjected to Shear," *ACI Journal*, ACI, Vol. 83, No.2, March-April, pp. 219-231.

Viest, I. M., Colaco, J. P., Furlong, R. W., Griffis, L. G., and Leon, R. T. and Wyllie, L. A., 1997, *Composite Construction Design for Buildings*. McGraw-Hill & ASCE, New York, USA.

Wakabayashi, M., Minami, K., and Komura, K., 1971, "An Experimental Study on Elasto-Plastic Characteristics of Concrete Members Using an Encased H-Section Subjected to Combined Bending and Axial Force," *Bulletin of Disaster Prevention Research Institute*, Kyoto University, No. 14A, pp. 417-437.

Wallace, J. W., 1989, *BIAX Users Manual: A Computer Program for Analysis of Reinforced Concrete Sections*, Department of Civil Engineering, University of California at Berkeley, CA, USA.

Weng, C. C., Yen, S. I., and Chen, C. C., 2001, "Shear Strength of Concrete-Encased Composite Structural Members," *Journal of Structural Engineering*, ASCE, Vol. 127, No. 10, pp. 1190-1197.

Weng, C. C., Yen, S. I., and Jiang, M. H., 2002, "Experimental Study on Shear Splitting Failure of Full-Scale Composite Concrete-Encased Steel Beams," *Journal of Structural Engineering*, ASCE, Vol. 128, No. 9, pp. 1186-1194.

Manuscript Received: May 22, 2006
Revision Received: Nov. 17, 2006
and Accepted: Dec. 20, 2006