Performance of Composite Concrete - Encased Columns of Full Interaction

Waleed A. Waryosh Ph.D (Asst.Prof.)* Mohammad Makki Abbass Bilal Ph.D (Lecturer)*

Saif Khudhair Ali Al-tameemi (Asst.Lecturer)*

Abstract

Composite column like reinforced concrete columns is a compression structural element made from a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally used to carry more loading as compared to concrete or steel section alone. The advantages of composite columns of concrete encased steel sections are high bearing resistance, high fire resistance, and economical solution with regard to material costs. Disadvantages are high costs for formwork, difficult solutions for connections with beams, difficulties in case of later strengthening of the column and in special case where edge protection is necessary. In this paper, analysis and design of composite columns with full interaction theory was adopted to check the performance and increased in columns capacity as compared to with columns made from concrete or steel section alone. ETAB software- based on the finite element method was adopted in the analysis and design its results where the compared to the results of a numerical result of a single column simulated in ANSYS with handout calculations to verify the analysis and design. Local and global buckling, capacity, and failure were checked for different boundary conditions supports at ends of each column.

Keywords: Composite Column, Full Interaction, Finite Elements, Buckling, Analysis and Design.

*Al-Mustansiriayah University

Introduction

In modern construction steel–concrete composite columns have been used worldwide owing to their benefits in carrying out huge loading and combining the rigidity and deformability of reinforced concrete with the strength of structural steel to produce an economic structure. In case of concrete-encased composite structural column the concrete used for encasing a structural steel increases its stiffness and protects it from fire damage and local buckling failure. Extensive researches on composite columns in which structural steel section are encased in concrete have been carried out. In-filled composite columns, however have received limited attention compared to encased columns. The behavior of the composite frame is different from that of a reinforced concrete frame, where the connection between beams and columns are usually monolithic. In 1999, El-Tawil, S and Deierlein, G. [1], studied the difference in nominal strength capacity requirements between ACI-318 and AISC-LRFD. In 2001, N.E. Shanmugam and B. Lakshmi [2], reviewed researches carried out on composite columns including experimental and analytical works. In 2005, João Batista Marques and Rodrigo Barreto [3], simulated a nonlinear numerical analysis of composite columns under axial force and biaxial moment. The results were compared to experimental data from literature and found to be close. In 2006, Brent S. and Robert G. [4], investigated the behavior of composite columns encased partially by using high performance concrete. The reference code adopted was CAN/CSA S16-01 that concern in normal weight concrete but in experimental tests they tested the composite column with and without eccentricity, and concluded that the design procedure adopted by code are conservative for high performance concrete. In 2006, Ehab M. Hanna and Abd El-Moniem M. [5], analyzed composite columns using finite element approach by Cosmos. Nonlinear analysis was used to investigate the strength of composite columns encased partially by concrete and the results were checked with experimental data from literature and showed close agreement. In 2007, P. Valach and Š. Gramblička [6], presented experimental tests result and theoretical analysis of composite columns. The columns were tested under concentric and eccentric axial loads and the tests results were compared to EN 1994-1-1. In 2011, Saima Ali and Mahbuba Begum [7], presented the behavior of partially encased composite columns partial encased under the effects of eccentric loading. They formulated load – deflection response by using Newmark's iterative procedure and concluded that the load eccentricity ratio has a significant impact on the capacity and deflection of composite columns encased partially by concrete. In 2013, Sharad.S and A.K.Gupta [8], studied the ultimate load capacity of composite columns by finite element approach. All simulation models results were compared to the available experimental results and showed good agreements. In 2013, Mote A.N. and Bhumkar Vijay [9], investigated and simulated composite columns encased in reinforced concrete. Both ends were pinned and the columns were subjected to axial loads only with different slenderness ratio values, then they were solved using nonlinear finite element approach. Comparisons between experimental and numerical solution indicated good agreements. The specification of ASCE, 2010 [10] are used in this paper.

In this paper, the performances of composite concrete- encased steel column sections by adopting full interaction theory assumed no slip between steel section and concrete. Finite element approach implemented by ETABS and ANSYS software was adopted with handout calculations to verify the analysis and design. Buckling, capacity, and failure were checked for different boundary conditions at ends of each column which were taken into account.

Strength of composite columns

An encased composite column is a column composed of a steel shape core encased in concrete partially or fully with additional longitudinal reinforcing steel and lateral ties to confine the main reinforcement bars. General composite columns parameters with symbols for encased type are shown in figure 1 and figure 2. According to the specification of AISC-2005 [11], the criteria that matching the specification requirements are, the cross-sectional area of the steel core which must comprise at least one percent of the total composite cross section, the minimum lateral reinforcement which must be at least (6 mm^2/mm) of tie spacing, and the minimum reinforcement ratio which is (4/1000) of the gross column area. The design compressive strength of composite column ($\phi_c P_n$) expressed as the ratio of Euler's formula derived according to column with pin ends (P_e) is as follows:

$$P_e = \pi^2 \, EI_{eff} \, / \, (kL)^2 \quad ...(1)$$

Equation (1) derived from (EIy'' = M), then the solution give homogeneous second order differential equation, of constant coefficients. Allowable compressive strength (P_n/Ω_c), where Ω_c is 2 (ASD) while the design compressive strength in (LRFD) is ϕ_c P_n, where $\phi_{c=}$ 0.75. The nominal axial compressive strength without consideration of length effects determined by taking summations of vertical forces and applying equilibrium equations, so:

$$P_o = A_s \, f_y + A_{sr} \, f_{yr} + 0.85 A_c \, f_c' \quad ...(2)$$

Where, A_s, A_{sr} , A_c , f_y, f_{yr} and f_c' are cross sectional area of steel section only, cross sectional area of concrete, area of continuous reinforcing bars, yield strength of steel section, yield strength of steel reinforcing bars and compressive strength of concrete, respectively. The strength of composite column denoted by (P_n) is calculated as follows:

$$P_n = P_e[0.658^{(P_o/P_e)}] \quad when \quad P_e/P_o \geq 0.44 \text{ ...(3)}$$

$$P_n = 0.877 P_e \quad when \quad P_e/P_o < 0.44 \text{ ...(4)}$$

The effective stiffness of composite section (flexural rigidity);(EI_{eff}) is:

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \text{ ...(5)}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_s + A_c}\right) \leq 0.3 \text{ ...(6)}$$

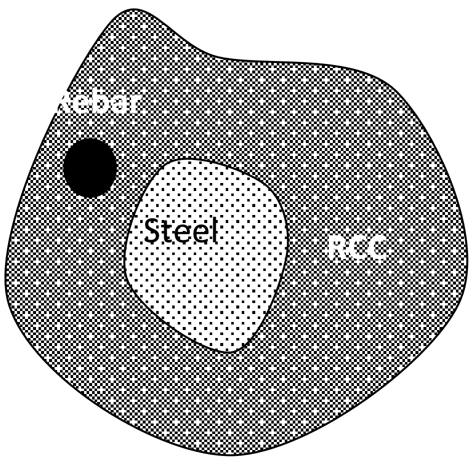


Figure 1: General parameters of encased composite column

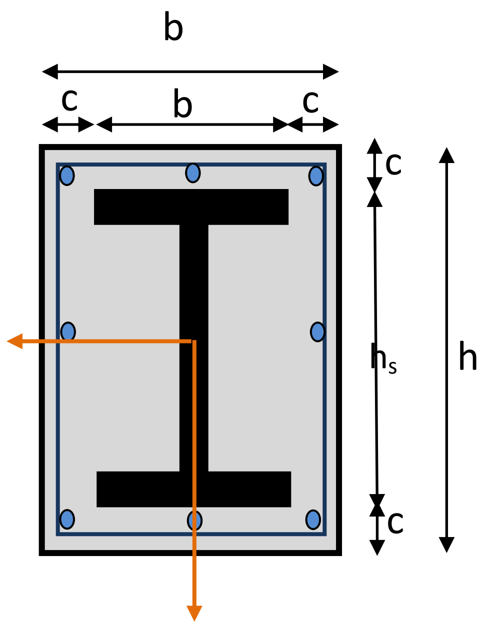


Figure 2: Encased composite column geometry.

Assumptions

In the present paper the following assumptions were adopted for encased composite columns analysis and design:

1. Full interaction between concrete surrounding the cross sectional area of steel section and steel section of main column (Full composite action up to failure)
2. Full bound between longitudinal bars and concrete.

3. The concrete is un-cracked.
4. The tensile strength of concrete is not taken into account, i.e. neglected.
5. Geometrical imperfection is equal to column height/1000.
6. The connection between composite columns and composite beams by steel sections.

Resistance of cross-section to compression

Design of composite columns consists of resistance of the member to structural stability, resistance to local buckling and introduction of loads. The interaction curve for a short composite column can be obtained by considering several positions of the neutral axis of the cross-section, (y_n), and determining the internal forces and moments from the resulting stress blocks. In the buckling resistance to compression for each axis of symmetry is first checked with the relevant non-dimensional slenderness of the composite column. Thereafter the moment resistance of the composite cross-section is checked in the presence of applied moment about each axis, with the relevant non-dimensional slenderness values of the composite column. For slender columns, both the effects of long term loading and the second order effects are included.

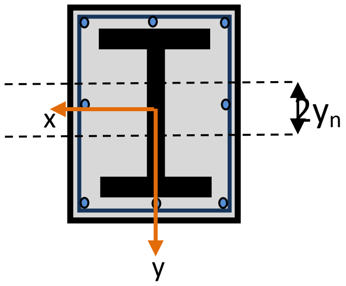


Figure 3: Major bending axis, the location of neutral axis of encased composite column

The plastic resistance to compression:

<u>Case1</u>, suppose that no bending moment effects the relationM_1=0) which is given by Eq.(2)

<u>Case 2</u>, corresponds to the plastic moment resistance of the cross-section (the axial compression is zero), the following relationship was derived, while the axial force is equal to zero:

$$M_2 = M_P = P_S(Z_S - Z_{S\,N/A}) + P_R\,(Z_R - Z_{R\,N/A}) + P_C\,(Z_C - Z_{C\,N/A}) \;\; ...(7)$$

Where Z_S, Z_R, and Z_C, are plastic section moduliof the steel section and reinforcement, concrete respectively while, $Z_{S\ N/A}$, $Z_{R\ N/A}$, and $Z_{C\ N/A}$ are plastic sections of the steel section, reinforcement and concrete about the natural axis, respectively.

Case 3, the compressive force and moment are as follow:

$$P_3 = A_C P_C \quad and \quad M_3 = M_P \quad ...(8)$$

There are two cases of composite columns layout, major axis bending and minor axis bending. In case of major axis bending location of the neutral axis is as follow:

$$y_n \le (h_s - t_f), so that\ y_n = \frac{A_C P_C - A_{Rn}(P_R - P_C)}{2b_C P_C + 2t_w (2P_S - P_C)} \quad ...(9)$$

Where A_{Rn} is the area of reinforcement within the range of ($2y_n$) percent.

$$(h_S/2 - t_f) \le y_n \le h/2, so that\ \ y_n = \frac{A_C P_C - A_{Sn}(2P_R - P_C) + (b_s - t_w)(h_S - 2t_f)(2P_S - P_C)}{2b_C P_C + 2b_S(2P_S - P_C)} \quad ...(10)$$

When the neutral axis is within the flange of steel section, then:
When the neutral axis is outside the steel section, then:

$$h_S/2 \le y_n \le h_C/2, so\ that\ y_n = \frac{A_C P_C - A_{Rn}(2P_R - P_C) + A_S(2P_S - P_C)}{2b_C P_C} \quad ...(11)$$

In case of minor bending axis the location of the neutral axis is as follow, Figure 4:

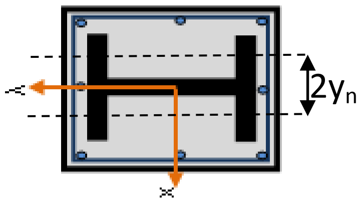


Figure 4: Minor bending axis, the location of neutral axis of encased composite column

The neutral axis within the web limit,is:

$$y_n \le t_w/2,\ \ so that\ \ y_n = \frac{A_C P_C - A_{Rn}(2P_R - P_C)}{2h_C P_C + 2h_S(2P_S - P_C)} \quad ...(12)$$

In case of neutral axis in flange within the range,

$$t_w/2 \le y_n \le b_S/2, so\ that,\ \ y_n = \frac{A_C P_C - A_{Rn}(2P_R - P_C) + t_w(2t_f - h_S)(2P_S - P_C)}{2h_C P_C + 4t_f(2P_S - P_C)} \quad ...(13)$$

In case of neutral axis outside the steel section,

$$b_S/2 \le y_n \le b_C/2, so\ that,\ \ y_n = \frac{A_C P_C - A_{Rn}(2P_R - P_C) - A_S(2P_S - P_C)}{2h_C P_C} \quad ...(14)$$

Local buckling

The nominal strengths for flexure and axial compression are dependent on the classification of the section as compact, non-compact, slender, or too slender. Compact sections are capable of developing the full plastic strength before local buckling occurs. The Encased I-Shapes are always considered to be compact [11].

Modeling

Numerical solution by the finite element approach using ETABS software was used to simulate thirty story high rise building to evaluate composite columns encased by concrete, they are shown in Figures 5 and 6. Various types of loading were considered, live load by assuming that the building is residential is evaluated according to ASCE – 7 – 2010 [10], Table 4-1, super imposed dead load and wind load are according to Iraq zone where the basic wind velocity is (45 m/sec) ,hence, the seismic load category A was adopted that match the Iraq requirements. Modal analysis was first run to determine the eigenvalues and eigenvectors. The eigenvalues are the frequency and eigenvectors are the modes where the first mode is the worst case assuming static behavior but under dynamic loading. The first mode is shown in Figure 7. The slab was (150) mm in thickness (filled slab), with stud shear connectors (19 mm) in diameter and (100) mm in height. The connection between composite columns and composite beams by steel section and then concrete encased steel columns section. Reinforced concrete slab with corrugated sheets working as composite and then this assembly with steel beams working another composite action and with assumption of full interaction like composite columns. Method of analysis adopted here was direct analysis – general second order and the design method was LRFD "Load Resistance Factor Design"- AISC 360-10 [11]. The Direct Analysis Method is more accurate in determining the internal forces of the structure, provided care is taken in the selection of the appropriate methods used to determine the second-order effects, and appropriate stiffness reduction factors

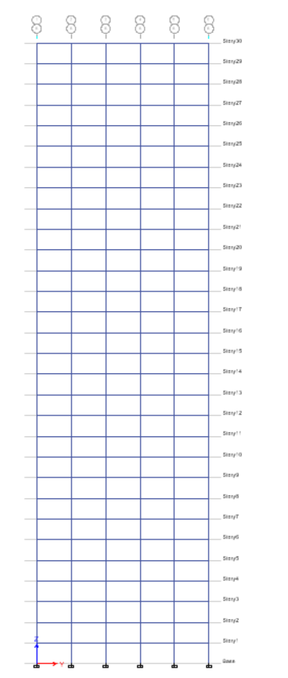
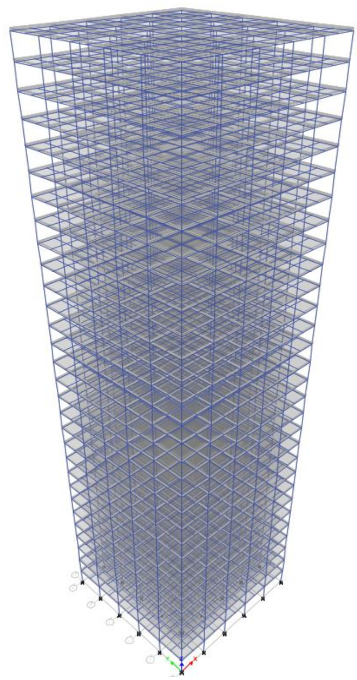
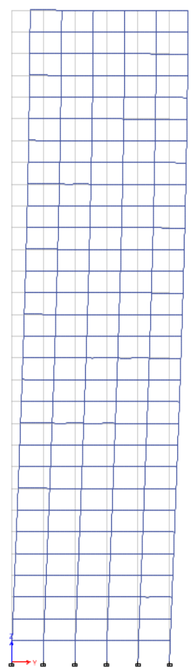
		

Figure (5): Elevation section.	Figure (6): Three dimensional section.	Figure (7): Modal-First mode.

The middle column that lies in the base was considered to evaluate the capacity, buckling, The Demand/Capacity ratios (D/C), determination of the controlling demand/capacity (D/C) ratios for each composite column member indicates the acceptability of the member for the given loading conditions. The columns dimensions are (350mmx350mmx3000mm) concrete cross section, the steel section HP250, total depth (250mm), flange (250mm), flange thickness (9.7mm), and web thickness (8.8mm), the compressive strength of concrete (25 MPa). The nominal axial compressive strength by applied equation (2) is (4575 kN), and the Euler's load is (27400 kN), with nominal tensile strength (2075 kN). The load capacity from ETABS analysis was used to apply this load when simulated composite column by ANSYS. Table (1), lists the modal periods and frequencies for the thirty stories high rise building adopted to check and evaluate encased composite column.

The maximum values of time (5.491) occur at the first mode that represents the worst case of modal case analysis because of work as static but under dynamic loading.
Table (1): Modal Periods and Frequencies.

Case	Mode	Period	Frequency	Circular Frequency	Eigenvalue
		sec	cycle/sec	rad/sec	rad^2/sec^2
Modal	1	5.491	0.182	1.1444	1.3095
Modal	2	5.287	0.189	1.1884	1.4124
Modal	3	4.18	0.239	1.5031	2.2593
Modal	4	1.812	0.552	3.4674	12.0226
Modal	5	1.742	0.574	3.6062	13.0043
Modal	6	1.394	0.717	4.5069	20.312
Modal	7	1.052	0.95	5.9713	35.6569
Modal	8	1.009	0.991	6.2266	38.7711
Modal	9	0.835	1.198	7.5275	56.6638
Modal	10	0.745	1.343	8.4388	71.2125
Modal	11	0.713	1.403	8.815	77.7037
Modal	12	0.596	1.677	10.5341	110.9677

In ANSYS model simulation Solid 65 was used for concrete, Solid 45 for steel section and Link 8 for main and stirrups reinforcement. Figure (8) shows the three dimensional modeling and Figure (9) shows the main and stirrup reinforcement. Two models were adopted fixed – fixed and fixed – pin. Table (2) and Figure (30&31) lists the maximum displacement for each case in three dimensions x, z and y.
Table (2) listed the maximum displacement for each case.

Boundary condition	Ux (mm)	Uy (mm)	Uz (mm)
Fixed – Fixed (I)	0.00165	0.0239	0.001466
Fixed – Pin (II)	0.0124	0.0195	0.001178

Because of restraint, there are differences in results between the two cases of boundary conditions.

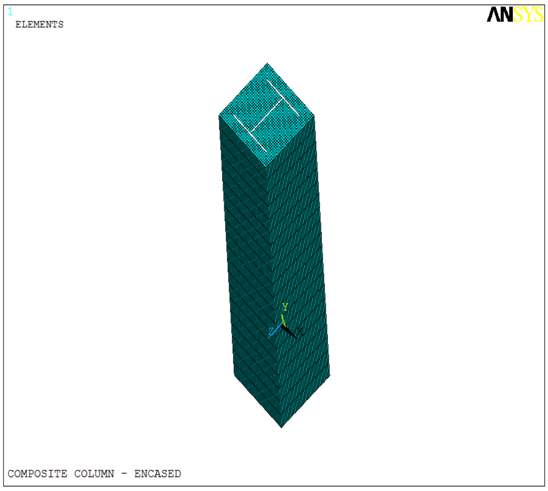
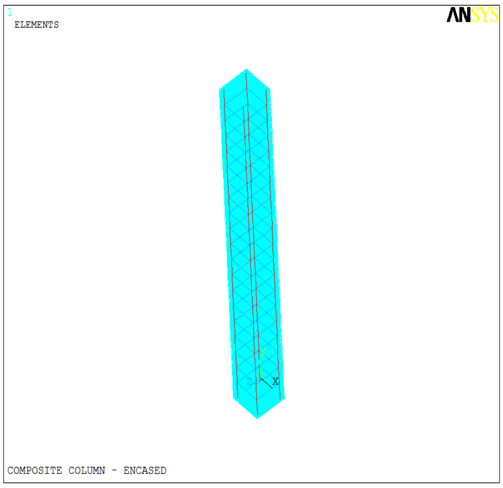
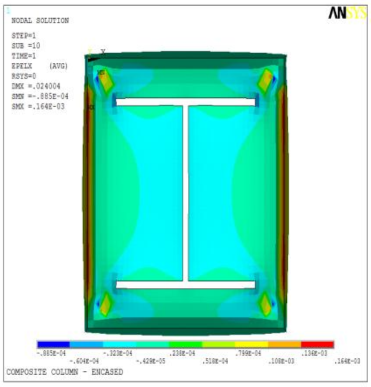
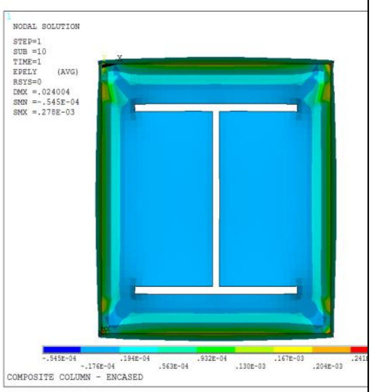
	

Figure (8): Elevation section.	**Figure (9):** Elevation section.

Figures (10) to (15), show variations of the elastic strain at the ultimate applied load in each horizontal direction (x,z) and the normal direction (y). There are differences in results because of different constraints at the top of column. Figures (16) to (21) show the displacement results at the final stage of loading. Figures (22) to (27) show the stress concentrations and distributions throughout the top cross sectional of encased composite columns. The stresses values in each direction compared with Von Misses criteria are shown in Figures (28) and (29), where they appear small.





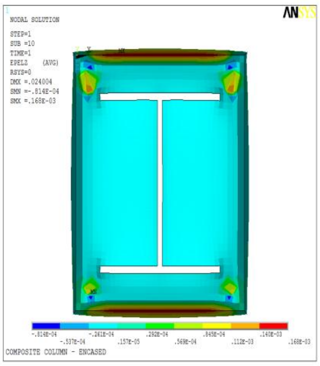


Figure (10): Elastic strain (x) - I.	Figure (11): Elastic strain (y) - I.	Figure (12): Elastic strain (z) - I.

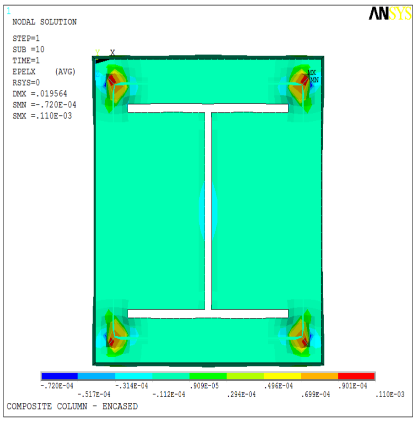
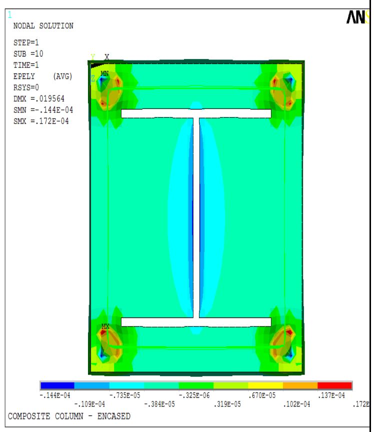
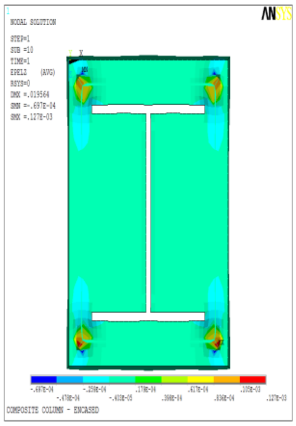

		

Figure (13): Elastic strain (x) - II.	Figure (14): Elastic strain (y) - II.	Figure (15): Elastic strain (z) - II.

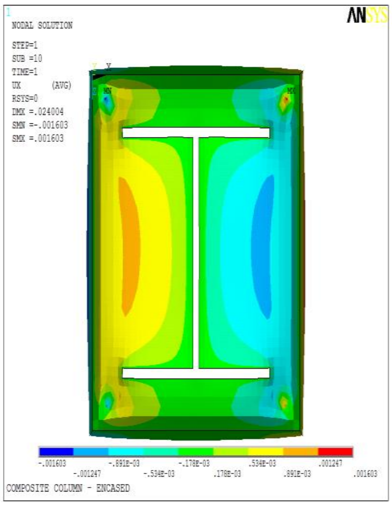
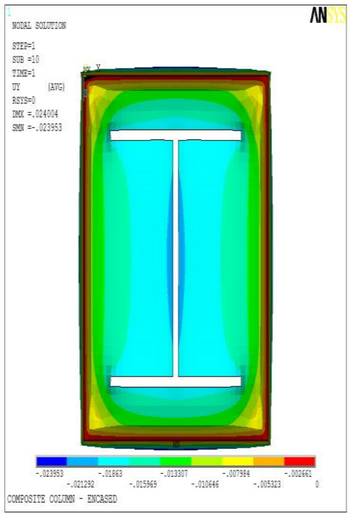
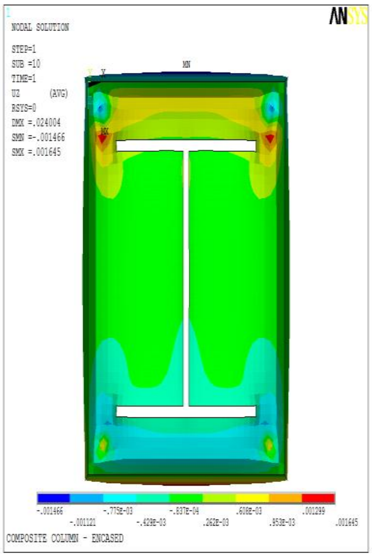

		

Figure (16): Displacement (x, mm) - I.	Figure (17): Displacement (y, mm) - I.	Figure (18): Displacement (z, mm) - I.

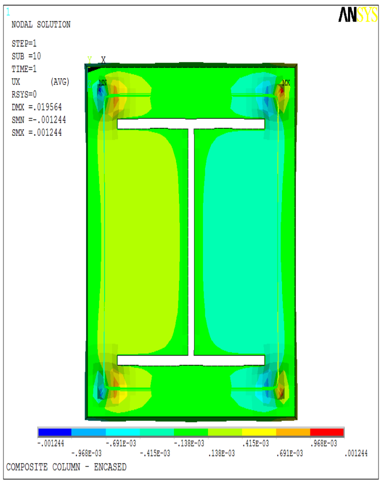
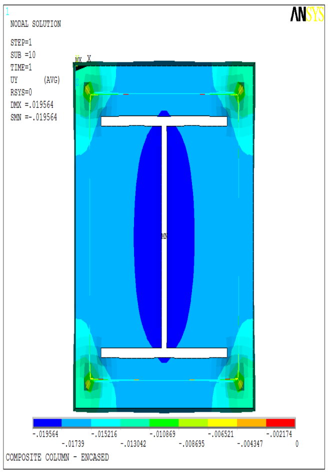
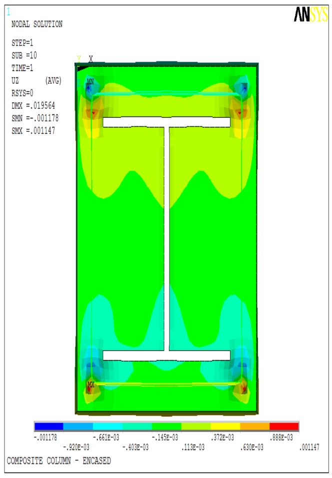
		

Figure (19): Displacement (x, mm) - II.	Figure (20): Displacement (y, mm) - II.	Figure (21): Displacement (z, mm) - II.

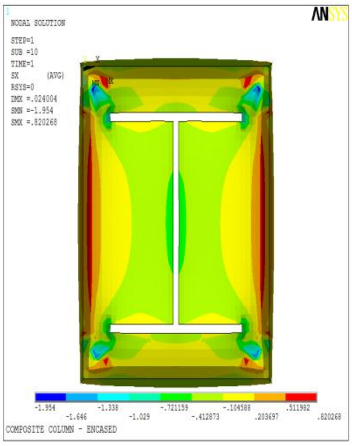
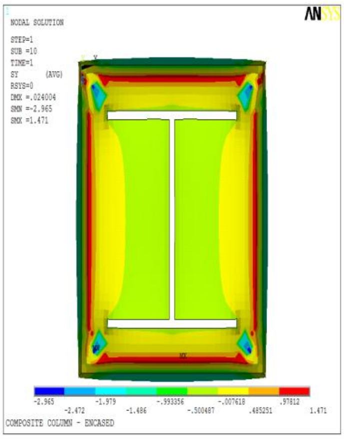
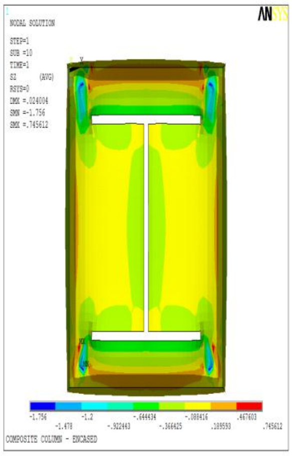

		

Figure (22): Stress (x, N/mm^2) - I.	Figure (23): Stress (y, N/mm^2) - I.	Figure (24): Stress (z, N/mm^2) - I.

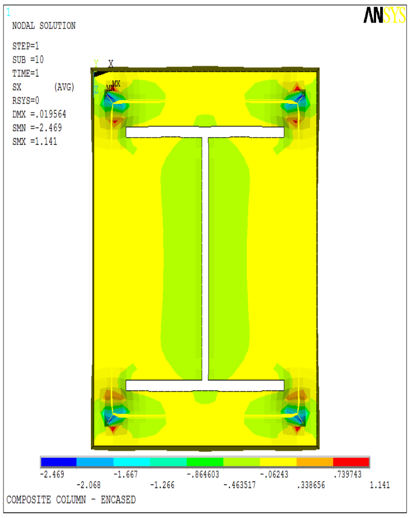


Figure (25): Stress (x, N/mm^2) -II.

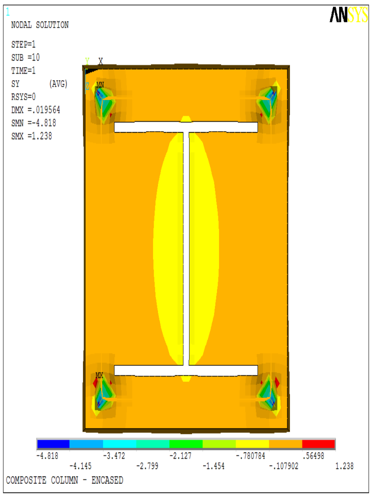


Figure (26): Stress (y, N/mm^2) -II.

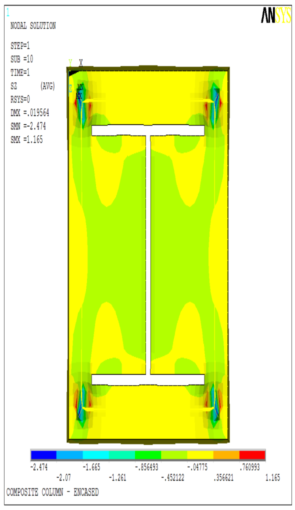


Figure (27): Stress (x, N/mm^2) -II.

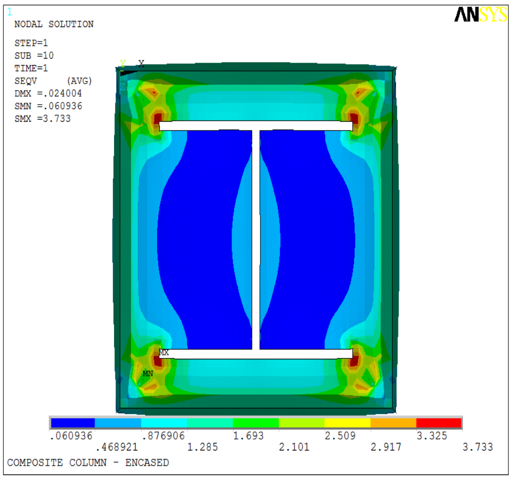


Figure (28): Stress (x, N/mm^2) –I – Von Misses.

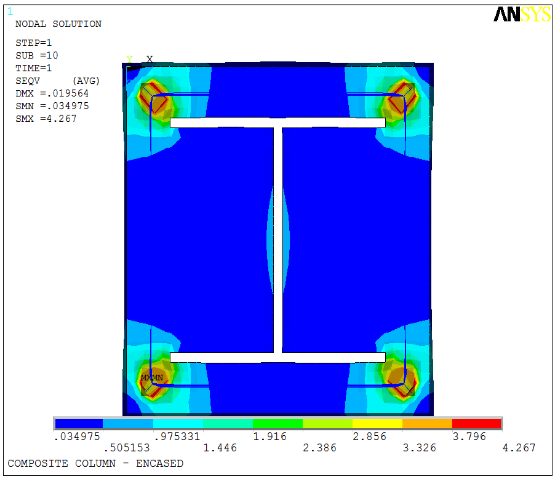


Figure (29): Stress (x, N/mm^2) –II – Von Misses.

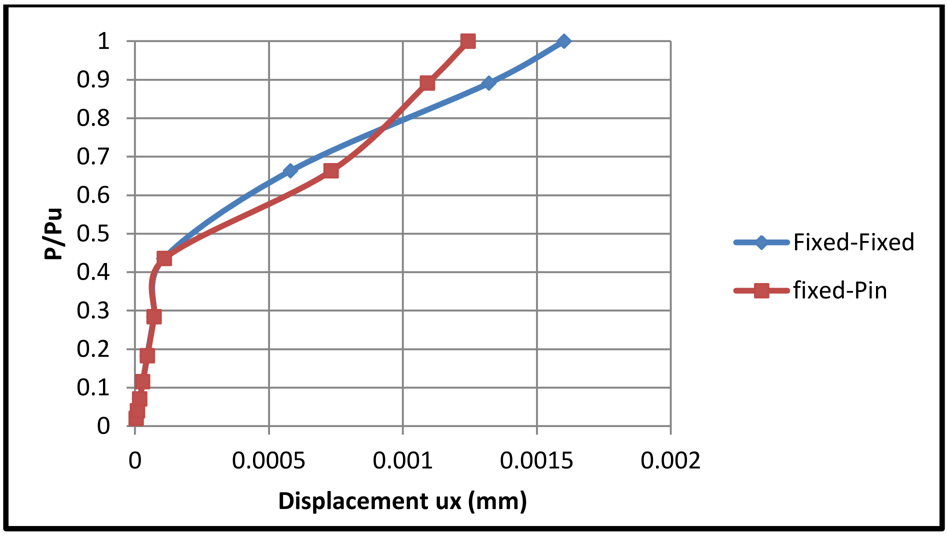


Figure (30): Displacement (ux)

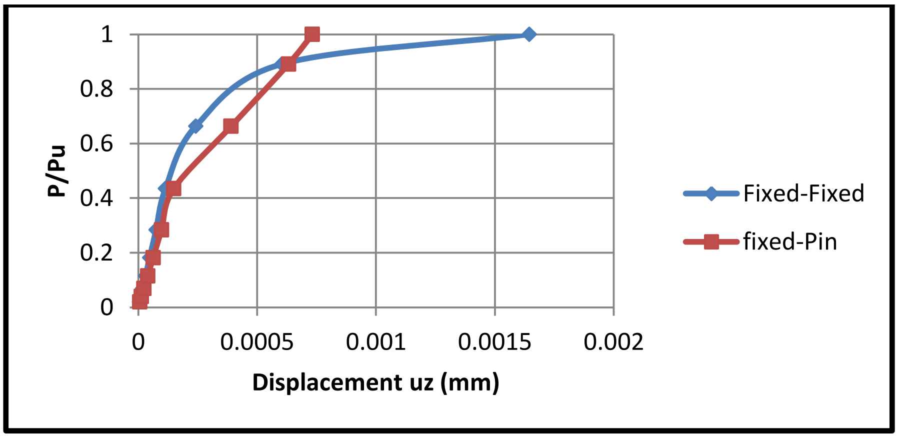


Figure (31): Displacement (uz)

Conclusions

The presence of the concrete is allowed for in two ways, to resist a small axial load, and to reduce the effective slenderness of the steel member which increases the resistance to axial load because the axial load capacity of composite column is larger than the steel section or concrete column alone. Followings are the most important conclusions that have been drawn from analysis and design of composite columns in full composite slabs, beams and columns subjected to various types of loading combinations:

1. Flexural stiffness is governed by concrete encasement
2. Encasement prevents buckling of steel bars and steel shape

3. Concrete outside ties cracks and spalls, followed by rest of encasement, checked by ANSYS by plotting concrete cracks.
4. After spalling, post-yield buckling of steel takes place, followed by overall failure.
5. Increased strength for a given cross sectional dimension.
6. Increased stiffness, leading to reduced slenderness and increased buckling resistance.
7. Significant economic advantages over either pure structural steel or reinforced concrete alternatives.
8. Identical cross sections with different load and moment resistances can be produced by varying steel thickness, the concrete strength and reinforcement. This allows the outer dimensions of a column to be held constant over a number of floors in a building, thus simplifying the construction and architectural detailing.
9. Formwork is not required for concrete filled tubular sections.
10. Concrete provides stiffening and strengthening.
11. This type of composite columns is used when exposed concrete finish is desired.
12. It is also used for transitions (concrete to steel columns).

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اداء الاعمدة المركبة المغطاة بالخرسانة ذات الترابط الكلي.

أ.م.د. وليد عواد ورَيوش*　　　م.د. محمد مكي عباس*　　　م.م. سيف خضير علي*

المستخلص

أن تصرف الآعمدة المركبة كعنصى انشائي يكون كتصرف الآعمدة الخرسانية المسلحة او الآعمدة الحديدية ولكن بتحمل اعلى. تستخدم الآعمدة المركبة لحمل السقوف العادية والمركبة وتكون ذات مزايا ممتازة من ناحية تحمل الاحمال, مقاومة الحريق, وذات تكلفة أقل من ناحية كلفة المواد. من ناحية اخرى, فإن هذا النوع من الاعمدة له سلبيات تتعلق بكلفة القالب, طريقة الربط مع العتبات, ويتطلب عناية خاصة حيث هنالك صعوبات في حالة تعزيز العمود وخاصة الاعمدة الواقعة في الحافة. تم في هذا البحث تحليل وتصميم الآعمدة المركبة من الخرسانة والحديد وتم اعتماد نظرية الترابط الكلي بفرض عدم وجود انزلاق بين المادتين لدراسة تصرف الاعمدة المركبة ومقدار تحملها مقارنة مع اعمدة خرسانية مسلحة واعمدة حديدية. تم الاعتماد على برنامج ETAB في التحليل والتصميم الذي يعتمد طريقة العناصر المحددة ومقارنة نتائج البرنامج مع نتائج الحل التحليلي وتحليل عمود منفرد باستخدام برنامج ANSYS لمعرفة التصرف. تم التطرق الى تدقيق انبعاج العمود المركب وقابلية التحمل بعدة شروط حدودية في النهايات.

* الجامعة المستنصرية