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Numerical Simulation of Concrete-Filled Steel Tubes Subjected to Biaxial Bending.

Abdel-Kareem Alrawdan
Faculty of Engineering., Mutah University Jordan.

NUMERICAL SIMULATION OF CONCRETE-FILLED STEEL TUBES SUBJECTED TO BIAXIAL BENDING

تحليل الأعمدة الخرسانية المركبة المعرضة الى عزم ثنائي حول المحور باستخدام طريقة المحاكاة العددية

Abdel-Kareem Alrawdan, Assistant Professor,
Faculty of Engineering, Mutah University,JORDAN

خلاصة

في هذا البحث تمت دراسة مشكلة الأعمدة الخرسانية المركبة و المعرضة الى عزم ثنائي حول المحور باستخدام طريقة المحاكاة العددية . بعض النتائج من التجارب المخبريه استخدمت لمعرفة سلوك هذا النوع من الأعمدة تحت تاثير العزم الثنائي, بحيث تم اخذ قيم مختلفة لبعد الحمل عن المركز ومساحة مقطع الاعمدة العرضي . الى جانب ذلك، النتائج التي تم الحصول عليها باستخدام الحاسوب تم تحليلها ومن ثم مقارنتها مع النتائج من التجارب المخبريه وايضاً مع الكود الأوروبي (EC4) . النتائج التي تم الحصول عليها تؤكد أنه يمكن استخدام طريقة العناصر المحددة غير الخطية كطريقة بديلة عن التجارب المخبريه المكلفة .

ABSTRACT

This paper addresses a numerical simulation of composite columns, in particular Concrete Filled Steel Tubes 'CFST' subjected to biaxial bending. Comparison with some experimental models is conducted in order to understand better the behavior of CFST and to validate the computation methodologies and results obtained. Results for various load eccentricity and different columns cross-sectional area are obtained, analyzed and discussed. In addition, the analysis considers material non-linearity of concrete and steel. The experimental results are also compared with the failure loads predicted in accordance with Euro-Code 4 (EC4). The results verify the applicability of the non-linear finite-element method as an economical and expedient alternative to expensive experimental work.

KEYWORDS

Concrete, Euro-Code 4, concrete-filled steel tubes, columns, finite element, biaxial bending

INTRODUCTION

As a result of the increase use of composite columns, a great deal of theoretical and experimental work has been carried out on both concrete-encased and concrete-filled composite columns. These investigations, for which comprehensive lists of publications are given elsewhere (1), have been developing since the beginning of this century. The work has culminated in the publications of BS5400 in the UK (4) and the European Recommendations for Composite Structures (6) by the European Convention for Constructional Steelwork

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(ECCS). Both methods of design are based on work by Basue & Sommerville (2&3) and also on a unified design method by Virdi & Dowling (15).

The design method in BS5400 covers both axially and eccentrically loaded composite columns, where the latter may be subjected to either equal or unequal end eccentricities. The design of a biaxially loaded column is also treated empirically in BS5400. Its carrying capacity, N_{xy}, is given by an expression which includes the column failure loads in uniaxial bending about both the major and minor axes, N_x, and N_y, and also the column failure load when loaded axially and restrained about its minor axis, N_{ax}.

Euro-Code 4 'EC4' (5) defines the requirements for the design of composite columns. The rules are valid for isolated non-sway columns. This includes columns of non-sway frames which may be considered to be isolated for the design. Two design method are given in the code; a general design method, which may be applied to any type of composite columns even to those with unsymmetrical cross-sections and a simplified method which is valid only for columns with double symmetrical sections and with constant sections over the column length. As the simplified method is assumed to be the method which mostly will be taken for the design, it will be used for the calculation of failure load in accordance with EC4.

Computer Aided Engineering 'CAE' tools enable several design and analysis iterations before first prototype hardware, and indeed permit the reduction of the prototypes required. It is estimated that, if $1 is spent fixing design problems in the CAE phase, that is equivalent to $10 in the prototype phase or $100 in the production phase. To keep concrete-filled steel tubes product development competitive, a system Finite Element analysis 'FE' approach to analyze and design the CFST was developed. The nature of the physical problem being such that, the behavior of each component of the CFST namely concrete and steel effect the performance of the other, so they behave as composite material. A simultaneous modelling and analysis of these interacting components was essential to achieve appropriate boundary conditions on each.

A method of analyzing the CFST is described here, having various nonlinear behaviors and subjected to a wide range of loads and eccentricities. It is believed that the method is sufficiently accurate to reduce the number of required prototypes and sufficiently fast to provide useful turnaround.

EXPERIMENTAL REFERENCE MODELS

In order to verify the correspondence of the numerical simulation with the experimental results and, subsequently, in order to find out simplified but equivalent numerical schematizations, the tests carried out on six full-scale composite columns will be reported. The columns are about 3m long which represent a typical storey height in multistorey buildings. Two columns were tested by Shakir-Khalil and Zeghichi (11) and four by Shakir-Khalil and Mouli (10). The steel section used was 120x80x5RHS. The eccentricities of the applied compressive force are summarized in Table 1 and also in Fig. 1, which together give the information relevant to the present work.

However, for the preliminary studies and comparisons, and in order to have a more direct knowledge of composite columns and to best approach the problems involved in the simulation of this kind, it has been decided to consider models with the same tested columns. In Table 1, the characteristics of the experimental tests executed are reported.

Table 1; Material Properties

Column No.	e_x (mm)	e_y (mm)	f_{sd} (N/mm^2)	f_{cu} (N/mm^2)	E_c (kN/mm^2)
H. Shakir Khalil and Zeghichi					
1	24	16	343.3	45	33
2	60	40	357.5	44	34
H. Shakir Khalil and Mouli					
3	12	8	341	42.6	35
4	42	28	341	46.3	34
5	24	40	362.6	42.4	33
6	60	16	362.6	40.8	33

GENERAL DESCRIPTION OF FE MODEL

General

After different preliminary calculations concerning the investigation of ABAQUS/S code capability of approaching the problem of interest, considering a reduced number of passes and making reference to the corresponding experimental tests previously presented, finally a calculation has been carried out for the simulation of the complete models.

The Friction Model

The classical Coulomb friction model (Aonton's law), with a limit on allowable shear stress provided in the ABAQUS program for use with gap and interface elements, was applied in this analysis. The model is based on the assumption that when surfaces are in contact, they do not slip until the shear stress tries to exceed the friction limit, which varies proportionally with the normal (compressive) stress acting between the contacting surfaces, up to the maximum shear stress limit. A detail account on the theory of this model is given in reference (7).

Loading

The load was applied on the column model at the top end which was considered to be free. As shown in Fig 2, the end load was applied at three nodes by force N_1, N_2 and N_3 such that their resultant compressive force acting on the column was applied at the required eccentricity. The summation of all loads acting at the nodes represented the total load 'N_n'. Control parameters are required to limit large increments in the loading. Loads were increased gradually throughout the time step, which started at zero and progressed gradually to one (end of step). The time steps chosen were very small which produced adequate information to derive the failure load for the column.

Boundary Conditions

As the column test specimens were loaded by equal end eccentricities, and due to the symmetry of the column, only half the column was modelled as shown in Fig. 3. The top end of the column model where the load is applied was considered to be free. The nodes at the column mid-height section are shown in Fig. 4, from which it can be seen that only central node 'A' is restrained in all directions. All the other nodes are restrained in the Z-direction, and only the nodes which lie on the X or Y axis, are further restrained in the orthogonal direction.

Material Properties of Steel

Steel is a ductile material, which exhibits non-linear material properties caused by plastic yielding and strain hardening. Material properties of steel tubes were determined from tensile tests performed in accordance with the specifications laid down in the British Standards. These non-linear material properties were assigned to the shell element used to discretize the steel tube model.

Material Properties of Concrete

The concrete core of the concrete filled steel tube has its own characteristics when the member is subjected to axial compression and bending. The stress state of the core concrete changed from uniaxial compression to triaxial compression as the load increased in the course. If the confining ratio ξ is large, the steel tube could offer effective confinement to its core concrete, and the bearing capacity of the core concrete can increase even if the stress of concrete reaches its failure criteria. However, if ξ is not big enough, the steel tube can not offer enough effective confinement to its core concrete, and after passing the peak point, stress-strain curves of the core concrete will continuously decline, the scope of declining varies with ξ. The bigger the ratio, the larger the scope of declining. The smaller the ratio, the less the scope.

Elastic-perfectly plastic behaviour, rather than strain softening, was assumed after the concrete has achieved f_{cu} in compression or f_{cu} /10 in tension. This was felt to be representative in compression, which is dominated by bearing behaviour of the concrete core. A measured average data of concrete compressive strength used in the concrete model was based on the cube strength. For tension, convergence problems resulted with a sharply softening tensile stress-strain curve; a maximum sustainable tensile stress of f_{cu} /10 was thus assumed.

Cracking In Concrete

Cracking is assumed to be the most important aspect of the behaviour, and representation of cracking and of post-cracking behaviour dominates the modelling. Cracking is assumed to occur when the stress reaches a failure surface that is called the "crack detection surface." This failure surface is a linear relationship between the equivalent pressure, p, and Von Mises equivalent deviatoric stress, q, and is illustrated in Fig. 5. When a crack has been detected, its orientation is stored for subsequent calculation. Subsequent cracking at the same point is restricted to being orthogonal to this direction since stress components associated with an open crack are not included in the definition of the failure surface used for detecting the additional cracks.

The model assumes that cracking causes damage, in the sense that open cracks can be represented by loss of elastic stiffness. It is also assumed that there is no permanent strain associated with crack. This will allow cracks to close completely if the stress across them becomes compressive.

The Mesh

Due to the symmetries, only half of the column has been modelized. The concrete core has been meshed with three-dimensional , twenty nodes continuum brick element 'C3D20'. The same mesh in C3D20 element has represented the bearing end plate where the load is applied. The rectangular steel tube has been meshed in eight nodes shell element S8R.

Interface contact element has been used to model the contact between the concrete and the inside surface of the steel tube. This element can be used to model a small sliding contact and separation between deformable bodies in stress/displacement simulations. This can be used in all three dimensional situations. These elements require matching meshes to be used on either side of the interface. Element connectivity needs to be defined.

MODEL, ANALYSIS AND RESULTS

The test results of the CFST when subjected to biaxial bending are discussed here, and this is followed by a comparison with the FE predictions, obtained by the application of the finite element program described earlier. Comparison will also be carried out between the experimental failure loads and the failure loads as predicted in accordance with EC4. The central deflection was recorded and plotted against the load, the full relationships of all the columns are shown in Figs 6 to 11. The figures show the severe effect on the displacements as the load and eccentricity increase.

The experimental, FE and EC4 predicted failure loads of columns are reported in Table 2, the values of the displacements corresponding to these loads are also reported in the same table. With the exception of column 3, the experimental failure loads of the columns are about 4% lower than the FE predictions with N_e/N_n mean value of 1.01. The predictions of EC4 are seen to be either higher or lower than the experimental values depending on the e_x/e_y ratio. The N_e/N_k varied between 0.91 and 1.10 with a mean value of 1.03. The table shows that EC4 predictions of the failure load are generally on the safe side, as they are lower than the experimental values.

Table 2; Experimental and FE Results

Column Number	e_x (mm)	e_y (mm)	Experimental			Finite Element			EC4	N_e/N_n	N_e/N_k
			Ne (kN)	δ_x (mm)	δ_y (mm)	N_n (kN)	δ^*_x (mm)	δ^*_y (mm)	N_k (kN)		
H. Shakir Khalil and Zeghichi											
1	24	16	268	12	26	261	11.1	30.5	251	1.03	1.07
2	60	40	160	21	36	154	19	32	159	1.04	1.01
H. Shakir Khalil and Mouli											
3	12	8	348	12	18.4	336	16	21	328	1.04	1.06
4	42	28	199	20.8	26.4	207	22	24	181	0.96	1.10
5	24	40	207	10.1	36.9	203	7.2	41	228	1.02	0.91
6	60	16	210	20.2	25.1	217	23	16.3	201	0.97	1.05

However, it seems likely that EC4 might yield unsafe designs for end eccentricities ratio e_x/e_y less than 1.0. The margin of safety increases when the increase of e_x/e_y. The experimental failure load of column 5 gives the largest percentage difference, on the unsafe side, when compared to the EC4 prediction, column 5 subjected to e_x/e_y ratio of 0.6. The FE predictions for the same column give reasonable estimate for the failure load.

The mode of failure of all columns followed an almost parallel course. Yielding in the compression corner of column mid-length section (bottom of the FE model), occurred at about 90% of the failure load. This was followed by yielding in the opposite corner in the tension zone.

Load-strain diagrams, obtained from the FE model, show that the non-linearity exhibited by strains is of the same nature as that exhibited by deflection. At failure, the recorded strains at column mid height varied between 1600 to 2000 μ strain. For columns with large eccentricities (columns 2, 4 and 6) the strain values were recorded at about 90 to 95% of the failure loads. The steel had yielded at failure both in tension and in compression and very high strains were recorded in the post failure stage (4000 to 6000 μ strain). Strain distribution along the column length obtained from the FE model agrees with that obtained experimentally. The contour plot showed that the strains were constant along the column length until a load level of about 90% of the failure load had been reached. At failure, however, strains recorded in the mid-length section were large than anywhere else along the column length as a result of the large lateral displacements at the column mid length section.

The experimental and FE load-displacement curves at mid-length were plotted for all columns, and Figs 6 to 11 show this relationship for columns 1 to 6. Both δ_x and δ_y are shown in the figure, and refer, respectively, to the lateral displacements in the directions perpendicular to the major and minor axes. The load-displacement curves show that, as expected, columns subjected to biaxial bending exhibited deformations in both directions. Comparing the FE load-displacement curves with those obtained experimentally, it can be seen that both curves exhibited non-linear relationships and are characterised by an ascending branch up to failure. The descending branch of the relationship was also recorded for all columns, the values of the displacements which correspond to the failure loads are reported in Table 2.

CONCLUSIONS

The test results of the experimental investigation reported here and the FE results show that EC4 yields conservative predictions for the failure loads of composite columns made of concrete-filled RHS and subjected to biaxial bending. It should be stated, however, that the margin of safety of such predictions seems to decrease with the decrease of e_x/e_y. Comparisons were only carried out on columns with maximum end eccentricities equal to half the depth of the columns. It is probable that the EC4 predictions might not be conservative in the case of columns subjected to end eccentricities of the same order as the column depth. Comparisons have also been made with the experimental mid-length displacements, and demonstrate that the FE models used are in good agreements with the measured displacement evaluations.

It can be concluded that, Finite Element modelling, with suitable material models, can be used to accurately predict the behaviour of composite columns made of concrete-filled steel tubes. The major features of a FE method of analysing CFST composite columns have been shown. The method is sufficiently fast and accurate to reduce the number of prototypes that would otherwise be necessary. The major nonlinear features of the CFST are successfully captured.

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NOTATION

e_x, e_y	Eccentricities of end force about major and minor axes
f_{cu}	Characteristic 28 day cube strength of concrete
f_{sd}	Design strength of structural steel, given by the respective characteristic strength divided by the material partial safety factor
N_e	Experimental failure load of column
N_k	Failure load of a column predicted on the basis of EC4
Nn	Numerically predicted failure load
δ_x, δ_y	Experimental column mid-length displacements at failure perpendicular to major and minor axes respectively.
δ^*_x, δ^*_y	Numerically predicted column mid-length displacements at failure perpendicular to the major and minor axes respectively.

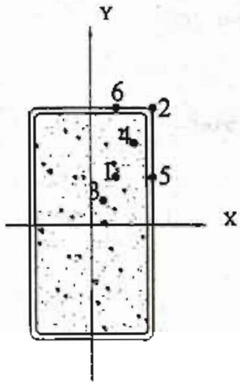

Fig. 1 Summary of Tested Columns

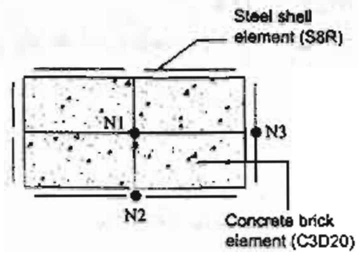

Fig. 2 Column Loading

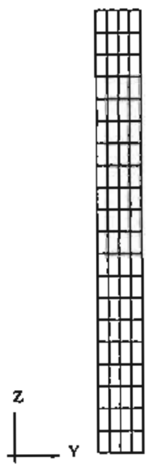

Fig. 3 Column FE Model

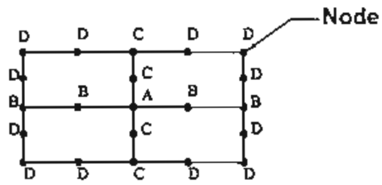

Fig. 4 Boundary Conditions at Column Mid-Height

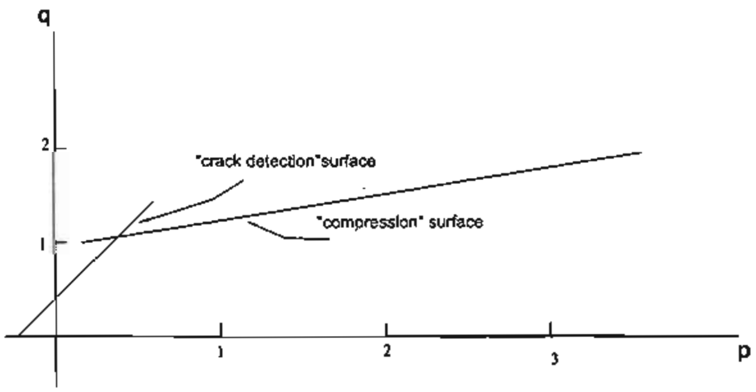

Fig. 5 Yield and Failure Surfaces in the (p-q) Plane

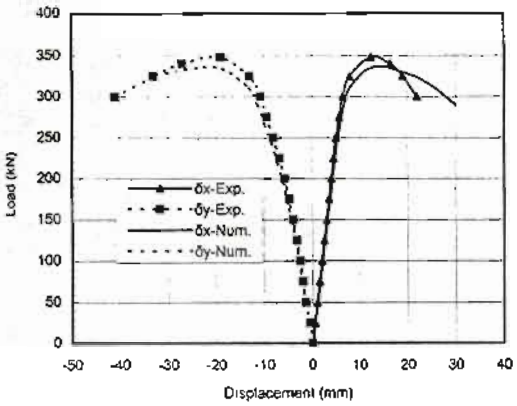

Fig. 6 Lateral deflections of Column 1

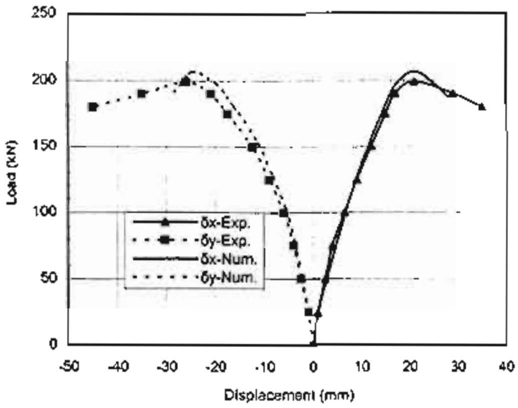

Fig. 7 Lateral deflections of Column 2

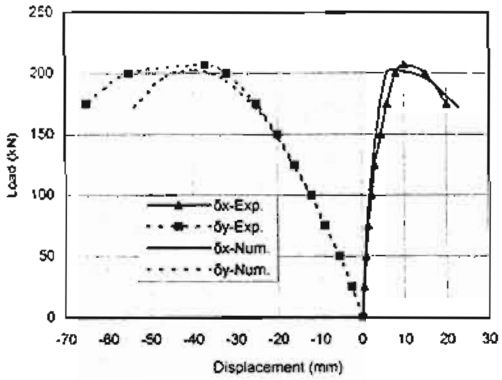

Fig. 8 Lateral deflections of Column 3

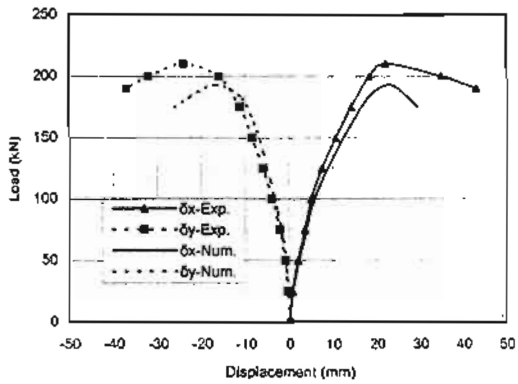

Fig. 9 Lateral deflections of Column 4

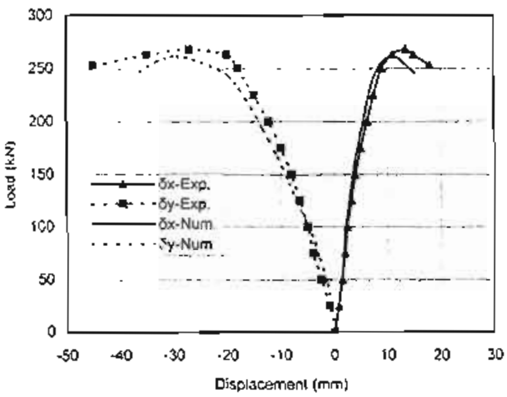

Fig. 10 Lateral deflections of Column 5

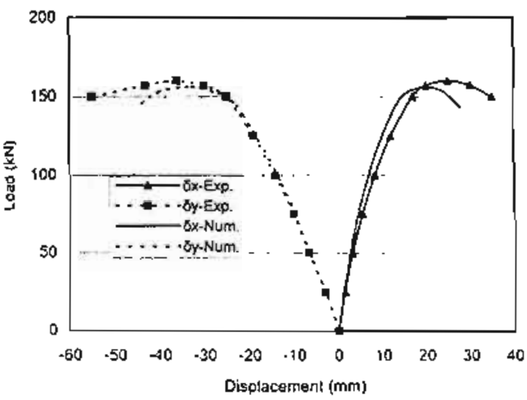

Fig. 11 Lateral deflections of Column 6