Prediction of Axial Load Carrying Capacity of RC Columns Retrofitted with Steel Jacketing

K.C.S. Gunarathna[1], B. Kiriparan[2], D.D.T.K. Kulathunga[3]

Abstract

Multi-storied building construction is one of the core component area in infrastructure development around the world. Addition and alteration to the existing buildings are involved commonly during the renovation of multistoried buildings. Overloading of existing columns beyond its carrying capacity is frequently encountered during the introduction of additional floors and alteration of intended floor function. Steel jacketing is one of the useful retrofitting techniques available for the strengthening of existing Reinforced Concrete Columns (RCC). Though steel jacketing is widely adopted internationally well-defined design provisions are not found in any of the design standards. The designs are either carried out using the provisions set out for composite columns or using experimental results. However, the behavior of the steel jacketed columns is significantly different from the composite columns. Further, various limitations are found even in the provisions given for composite columns. In the design of steel jacketing and composite columns width to thickness (b/t) ratio are limited to prevent local buckling of steel plate. Design provisions for slender sections are not covered in the international standards such as British and European standards. This paper intended to present the local buckling behavior and axial load carrying capacity of steel jacketed RCC columns using numerical simulations.

1. Introduction

Multistory building construction is one of the core components in the infrastructure development around the world. Addition and alteration to the existing building involved commonly during the renovation of a multistory building. In a multistory building structure, a column is one of the key elements that bear and transfer the loading to the ground. Overloading of existing columns beyond their carrying capacity is often encountered during the introduction of additional floors and alteration of the intended usage . A few such scenarios are as follows; 1. Replacement of lightweight roof with a concrete slab in low rise buildings 2. Introducing additional floors on existing buildings to cater to the owner's requirements 3. Alteration to the design scheme by the Client after partial completion or during the construction 4. Alteration to the intended floor function converts residential buildings to offices or industrial floors, introducing mechanical equipment, water tanks, pools and such cases which was not intended in the design. Several strengthening methods are developed internationally to overcome this concern. A few of such commonly adopted techniques for the strengthening existing columns are; 1. Concrete jacketing, 2. Steel jacketing, 3. Precast concrete jacketing, 4. External prestressing, 5. FRP strengthening.

All the above-mentioned strengthening techniques have their advantages and limitations. Among them, steel jacketing is becoming increasingly popular due to reasons such as; 1. Relatively minimal loss of usable space around the columns, 2. Minimum disturbance to the adjacent structural/nonstructural elements, 3. Suitability to obtain higher capacity increment ratios (e.g. more than 200 %) 4. Less consumption of time and improved constructability 5. Less work on the preparation of columns for strengthening 6. Better performance under earthquake and blast loadings[1].

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2. Literature Review

For the designing considerations of steel jacketing no properly defined guidelines are available. Hence, the codes for composite structures or test based capacity assessments are in use for this purpose.

One such experimental based suggestion for the thickness of steel jacket for square RC column is given in equation 1 [2]

2.1 Test Based Assessments

The thickness of steel jacket for square RC column,

$$\boldsymbol{t_j} = \frac{\boldsymbol{f_{co}}}{\boldsymbol{f_{jy}}} \times \frac{\boldsymbol{B\sqrt{2}}}{\boldsymbol{8k_f}} \quad ......(1)$$

Another experimental investigation done by Richard et al. suggests the expression given in equation 2 for the confined concrete strength for steel jackets that do not extend to full height.

$$f_{cc} = f_{co} + k_1 f_1 \quad ......(2)$$

for fully confined columns (ACI 318M- 99);

$$P_o = 0.85\left(A_s f_y + A_j f_{jy} + 0.85 A_c f_c\right) \quad ......(3)$$

Most commonly steel jacket is designed by considering it as Concrete Filled Steel Tube (CFST) column, which can be referred to as a composite column design in many internationally recognized codes. Provisions for composite column design are included in codes such as ECP 203-200, ECP-Sc-LRFD-201, ACI-318-08, AISC-LRFD-2010, BS 5400-Part 5, Chinese code CECS159, Hong Kong code, and EN 1994-1-1:2004[3]

2.2 Codes for Composite Structures

As per BS 5400 – part 5 (British standard: code of practice for design of composite bridges), wall thickness of a rectangular hollow tube and the ultimate axial load carried by the composite column can be expressed as shown in equation 4 and 5. Here, t is the thickness of the wall, b_s is the external dimension of the wall of the RHS, f_y is the nominal yield strength of the steel, E_s is the Young's modulus of steel.

Wall thickness (rectangular hollow)

$$t \geq b_s\sqrt{f_y/3E_s} \quad \text{...... (4)}$$

axial load at ultimate limit state;

$$N \leq 0.85K_{1y}\left(0.91A_s f_y + 0.87A_r f_{ry} + 0.45A_c f_{cu}\right) \quad \text{...... (5)}$$

Further, as per the recommendations given in Eurocode 4 (Design of composite steel and concrete structures), ultimate load of concrete filled steel tube can be expressed as given in equation 6. Here, Aa is cross-sectional area of reinforcement, f_{yd} is the yield strength of reinforcement, Ac is the cross-sectional area of concrete, f_{cd} is the yield strength of concrete, As is the cross-sectional area of steel and f_{sd} is the yield strength of steel.

$$N_{pl,Rd} = A_a f_{yd} + A_c f_{cd} + A_s f_{sd} \quad \text{...... (6)}$$

Due to the inconsistency of design codes, comparative studies have done to identify the differences. Some such studies and their findings are quoted here. By comparing design scopes of the JGJ138-2016 and simplified method in Eurocode 4, [4] Q. Zhang et al states that Eurocode 4 allows the design of small size columns as well as the design of irregular composite columns whereas JGJ138-2016(Chinese standard: code of design for composite structures) only suitable for larger cross-sections. Due to the use of higher partial safety factors on materials and allowing the use of high strength material Eurocode 4 gives comparatively higher design strength value. As such the considerations in the two codes are conservative. In the comparison of AISC-LRFD (American standard: load and resistance factor design method) code with Eurocode 4, it showed that AISC- LRFD is more conservative.[5]. The method proposed by M. Abramsky [6] is in agreement with the Polish code and capable of providing a higher safety than the Eurocode 4 design method. Further, a comparative analysis of different guidelines for design steel encased concrete structures done by Amiya and Amit [7] proves that the evaluated results on the strength of the column varies widely in Eurocode 4, ACI Code and AISC-LRFD and not similar to the INSDAG (Institute of steel development & growth, India) model applied in the Indian context.

By referring to past researches on steel jacketing technique and CFST columns, several mathematical models have been suggested to calculate the strength of the CSFT column but are very conservative. At the same time, there's an issue at hand regarding the applicability of those provisions for the design of steel jacket in the retrofication. Therefore, this study was done to capture the variations of axial load carrying capacity of RC column with a steel jacket under different parametric conditions through a Finite Element Model (FEM) analysis. The results were then compared with the available design guidelines to find their applicability on the steel jacketing technique and a mathematical model was suggested to be used as an initiation for preparing a well-defined provision to use when designing the steel jacket for retrofication.

Generally, there are two ways of assembling the steel jacket to the existing RC column. One way is to use shear connectors and the other way is to use Epoxy bonded steel plate. A shear connection can be achieved by shear studs, shear rings or shear plates. However, in this study the focus is only on epoxy bonded steel plates.[8]. Within the scope of this research, the steel jacket is modeled as it is glued to the RC column and the parametric study was carried out only for the axially loaded isolated square columns.

3. Methodology

For the parametric study, specimens were modeled using ABAQUS 2019. As the first step, only the steel tube was modelled. Eigenmode values for the model were obtained by a linear perturbation step and then nonlinear buckling analysis was carried out for the improved model with plasticity characters of the material. These numerically obtained axial load capacity values were compared with the experimental values reported by B. Uy [9]. For one selected model a mesh convergence analysis was carried out to find the optimum mesh size, to obtain more accurate results within a less computational time. Then the concrete core was modeled. Required input material parameters were obtained after referring to many works of literature on the confinement effect of concrete. The composite model is then validated by comparing the experimental results reported by Uy [9]. Then a parametric study was done by varying the cross-section dimensions for slender and stub columns. Finally, the results were compared with the predictions of the BS code and Euro code and were able to provide a summary of the analytical results.

3.1 Modelling of Steel Tube

The specimens were modelled referring to the experimental study by [9]. There the concrete filled steel box columns of different cross-sections were experimented about the local and post buckling behavior. The specimens were arranged to be in two sets as columns loaded only on the steel and the other to load uniformly over the composite section. A steel tube without concrete core was modeled initially in this study and validated the model simulations with the experimental results. For the validation of model which has only the steel tube, steel tubes which has dimensions given in Table 1 and stress/strain model in Table 2 was used.

The steel tube was modelled as S4R element type and given the boundary conditions as top in the type of displacement/rotation and bottom to be encastre. General/static/riks analysis was done using the eigenvalues resulted in the linear perturbation step of linear buckling analysis. Force vs Displacement curves was obtained, and the ultimate capacity was compared with experimental results. These results are plotted in Figure 1 for the two types of steel tubes used which proved the numerical models are of acceptable accuracy.

Table 1: Specimen dimensions and properties

	LB2	LB10
Column height (mm)	300	3000
B (mm)	126	306
b (mm)	120	300
t (mm)	3	3
b/t	40	100
Yield stress (MPa)	300	300
Ultimate stress (MPa)	410	410
E_S (Gpa)	200	200
Poisons ratio	0.3	0.3

Table 2: Stress/strain model of steel

Stress (MPa)	Strain
300	0
410	0.3

3.2 Modelling of Concrete Core

In order to model the composite column, a new part was introduced to the verified steel tube model, and the concrete core was modelled in the element type of C3D8R. The compressive strength of concrete used in the experimental specimen was 40MPa. In general, the Poisson's ratio of concrete ranges from 0.15 – 0.22 and in this study, it was assumed to be 0.2 for uncracked concrete[10]. The elasticity modulus of confined concrete was calculated using Equation 7 by using the unconfined compressive strength of concrete since almost all relatable modelling have used the mentioned equation. For unconfined concrete model Equation 8 was used.

$$E_C = 4730\sqrt{f_c} \quad ......(7)$$

$$E_{CU} = 5.5\sqrt{\frac{f_{cu}}{1.5}} \quad ......(8)$$

With the confinement by steel, the yield stress of concrete increases as the concrete core undergoes triaxial stress [11]. To model this behavior concrete damaged plasticity (CDP) model was used since it accommodates uniaxial stress-strain on unconfined concrete with adjustments for the confinement. In the CDP model, the following data need to be entered. Dilation angle (ψ), flow potential eccentricity (ϵ), the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress (fb/fc), the ratio of the second stress invariant on the tensile meridian (K) and viscosity parameter (μ). Also, compressive and tensile behavior of the modelling material need to be defined. For this study dilation angle proposed by Y.R.Abbas [11], which is given in Equation 9 was used.

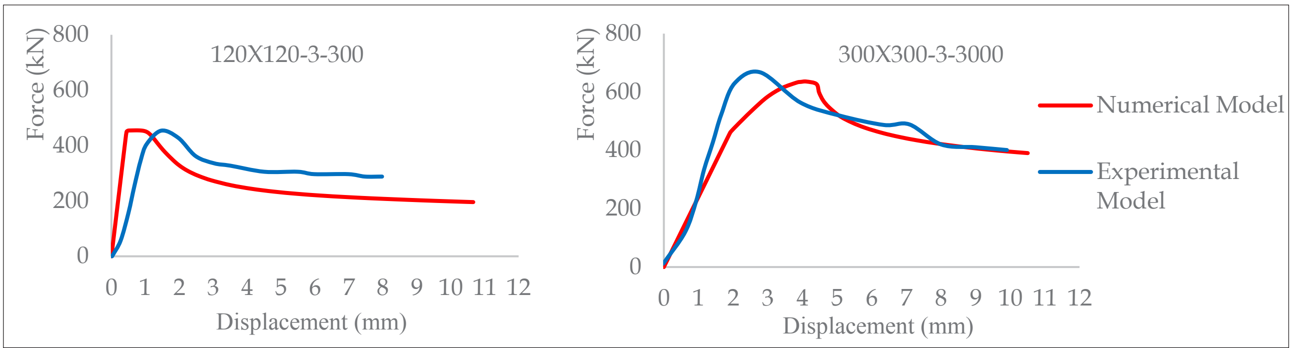


Figure 1: Model validation for steel tube

$$\psi = \begin{cases} -2.0769 * \varepsilon_c^2 - 42.396 * \varepsilon_c + 63.72, \\ \qquad ;\ f_y \leq 400 \\ -0.0138 * C^2 - 0.0625 * C^2 - 9.558 * C + 71.49, \\ \qquad ;\ f_y > 400 \end{cases} \quad ......(9)$$

$$\varepsilon_c = \frac{A_S * f_y}{A_C * f_c}, \qquad C = \varepsilon_c + 1.9^{\frac{f_y}{200}}$$

Here ε_c is the confinement factor, A_S and A_C are the cross-sectional area of steel and concrete respectively, f_y the steel tube yield strength, f_c concrete core compressive strength. It was said to use a value 56^0 if the calculated dilation angle is higher than $\psi=56.3^0$. The ratio fb/fc was calculated using Equation 10 recommended by V. K. Papanikolaou and A. J. Kappos [12]).

$$\frac{f_b}{f_c} = 1.5 * f_c^{-0.075} \quad ......(10)$$

To determine the value for (K), the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield for any given value of the pressure invariant, ABAQUS manual has given range of values from 0.5-1 and a default value K = 2/3. But according to the study by Z. Tao et al [13], the influence of K is significant after yielding though it's not before yielding. Hence the Equation 11 is used to determine the K value.

$$K = \frac{5.5}{5 + 2(f_c)^{0.075}} \quad ......(11)$$

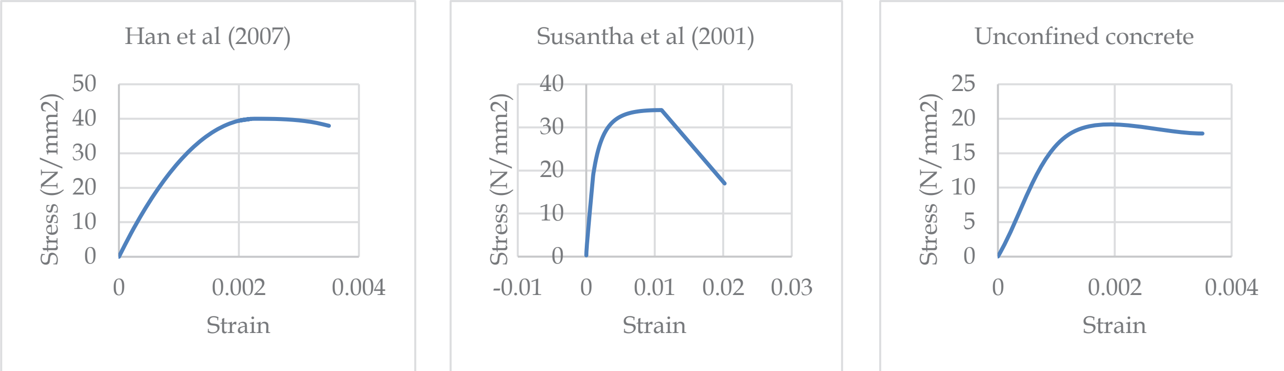


Figure 2: Stress/Strain model for concrete in compression

The values taken for viscosity (μ) and flow potential eccentricity (∈) are 0 and 0.1 respectively. The compressive behavior of the concrete was defined using three material models, two confined material models by L.H. Han et al [15], K. A. S. Susantha et al [16] and the unconfined model of BS 8110 [17] as in Figure 2. The results by using mentioned compressive behavior was compared with the experimental results to validate and then to choose the most suitable compressive behavior curve to model the concrete core.

Generally, in defining concrete material properties, the tensile behavior of concrete is not taken into consideration. But the CDP model in Abaqus uses tensile behavior parameters also to define the material. Here it provides the option of defining the tensile behavior in three types, yield stress vs crack strain, yield stress vs displacement and yield stress vs fracture energy (GFI). A mathematical model to define yield stress vs crack strain was developed by B. Alfarah et al [18] for unconfined concrete. The procedure was followed in conjunction with fracture energy criterion proposed by Z. Tao et al [13]. Accordingly, in the current model, it was assumed that tensile behavior to be linear until the tensile strength given as $0.1f_c$. and beyond that, curve was defined according to the mathematical model by B. Alfarah using Equation 12 to define fracture energy (G_F). Figure 3 is the tensile behavior of concrete used in this study.

$$G_F = (0.0469d_{max}^2 - 0.5d_{max} + 26)\left(\frac{f_c}{10}\right)^{0.7} \text{N/m} \;......(12)$$

Here d_{max} is the maximum size of coarse aggregate in the concrete mixture and said to be use 20mm if no mention in the references

3.3 Surface Interactions

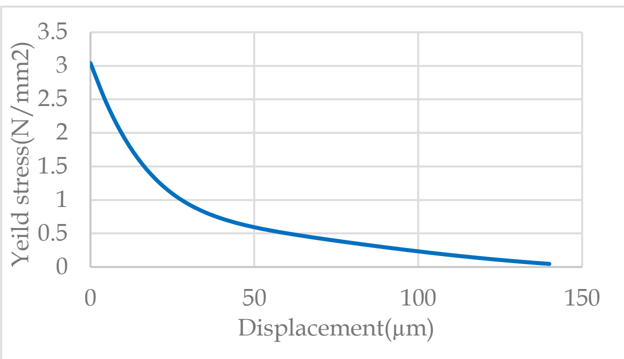


Figure 3: Concrete material behavior in tension

The interaction of the steel and concrete surfaces was modelled using surface-surface contact. There the normal behavior of the two surfaces was set as hard contact assuming that concrete surface doesn't penetrate the steel surface. And to define the tangential behavior penalty friction formulation method was used. Assuming the steel jacket was glued to the column using epoxy glue, o.5 was used as the friction coefficient between the surfaces.

The composite model with the three different compressive behavior models was then validated against the experimental results. The validation curve is in Figure 4 and as of that the 1182kN of the capacity is the most similar to the experimental result of 1131kN. Therefore, unconfined material properties were used for further modelling of the concrete core.

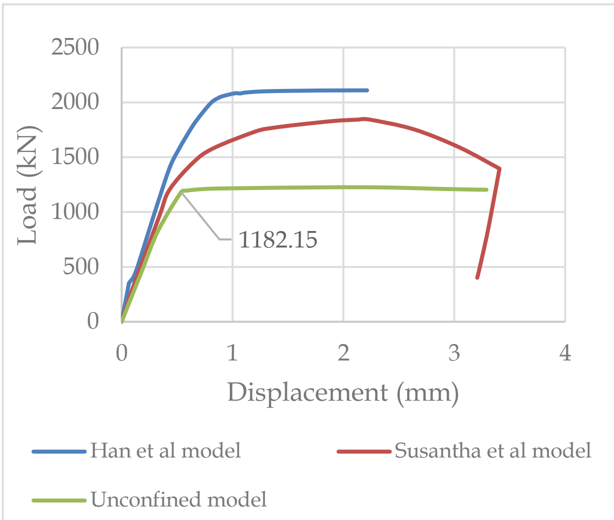


Figure 4: Composite column variation

3.4 Mesh Convergence Analysis

The identification of the optimum mesh size is important in numerical simulations to obtain adequate output from the simulation. The accuracy of the results drops as the refinement of the mesh is less. As stated in the Abaqus user guide, after one refinement of the mesh, the results tend to be converged with the increase of the mesh refinement. To identify this optimum mesh size, a mesh convergence analysis was carried out with 5mm, 10mm,15mm and 20mm mesh sizes for steel tube columns and 20mm,25mm,40mm, and 50mm mesh sizes for composite columns. Figure 5 is the mesh convergence curves for steel tube and composite column and uses the 20x20 mesh size for steel tube and 40x40 mesh size for concrete core in the models for the parametric study.

4. Parametric Study

The main purpose of this study was to develop a numerical model of the RC column retrofitted with a steel jacket to predict the axial load carrying capacity. Whereas the variation of the capacity of the column with the change of cross-section size, column slenderness, and jacket thickness was analyzed.

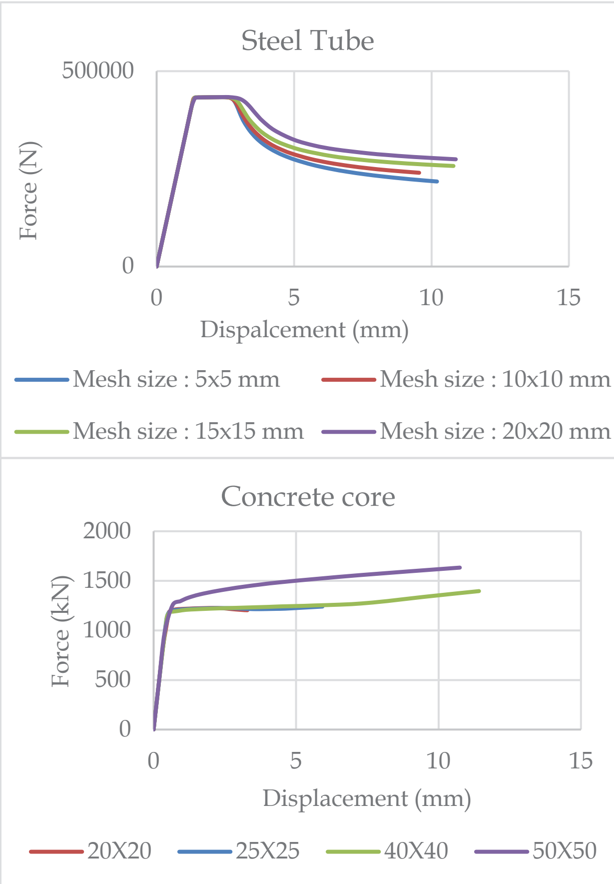


Figure 5: Mesh convergence analysis

Therefore, to capture these variations, the validated model of the composite structure was modified by changing the cross-section, height and jacket thickness. The final objective of the study was to give recommendations on the use of the standard design codes for the capacity prediction of steel jacketed columns. 16 nos. of models were developed for the analysis. The models being referred to as of the index in Figure 6.

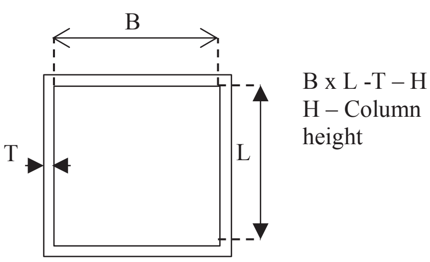
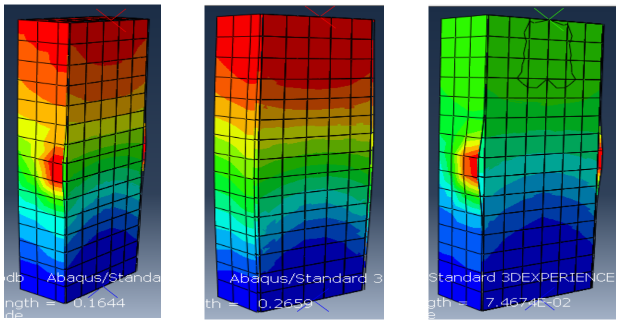


Figure 6: Model index



100x100-3-300 100x100-5-300 120x120-3-300

Figure 7: Parametric models after nonlinear analysis

Figure 7 is the cross section view of parametric models after nonlinear analysis.

5. Results

The initial observation of the analysis as of Figure 8 was the effect of steel confinement on the RC column considering the axial load capacity. Placement of the jacket has caused a capacity increment around 60% have resulted here.

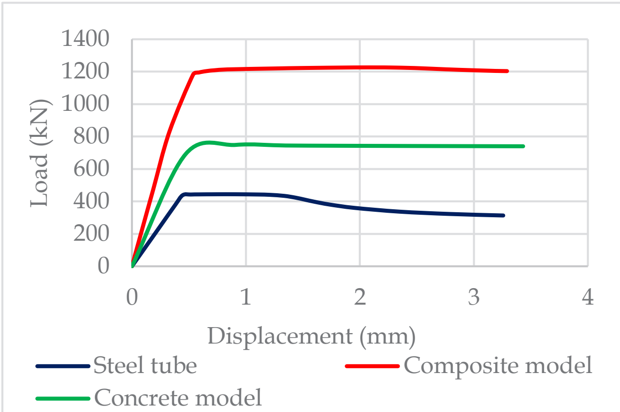


Figure 8: Effect of steel jacket on the capacity of the column

5.1 Effect of Variation of Cross Section Size

The comparison of the results for different column heights and cross-sections is shown in Figure 9. It can be seen from the figure that, the column with the higher cross-sectional area of concrete has higher axial load capacity.

5.2 Effect of Variation of Jacket Thickness

Variation of jacket thickness was checked against two models of 3mm and 5mm jacket thickness and the results are shown in Figure 10. The figure shows that the slight increase in the thickness of the steel jacket which is from 3mm to 5mm has caused capacity increment by 26% - 28%.

5.3 Effect of Variation of Column Height

There was no change in the axial load carrying capacity with the increase of the column height for the plastic sections where b/t ratio less than 40. But the stiffness of the columns was reduced significantly showing comparatively higher displacement at the peak load as in Figure 11.

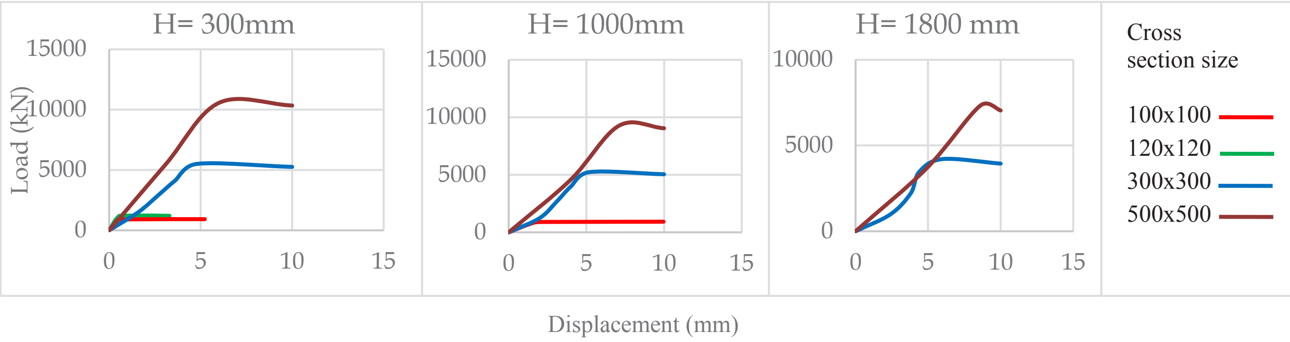


Figure 9: Effect of variation of cross section size column

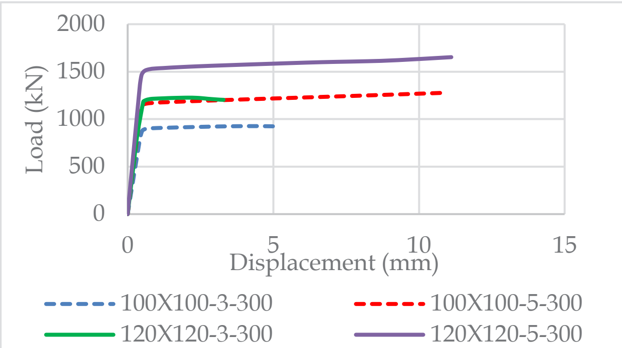


Figure 10: Effect of variation of jacket thickness

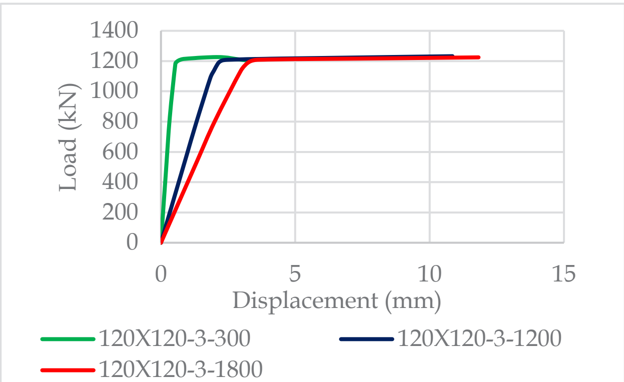


Figure 11: Effect of variation of column height on axial load carrying capacity of columns with plastic c/s

For columns with slender sections where b/t ratio greater than 40 both capacity and stiffness of the column showed a decrease with the increase of column height. This variation is compared in Figure 12.

5.4 Effect of Slenderness on the Failure Load of the Column

Variation was similar to concrete columns where failure load reduced with the increment of slenderness ratio as shown in Figure 13. However, for columns with plastic cross-section significant variation was not seen in the failure load with the increment of slenderness ratio.

5.5 Effect of b/t Ratio on the Failure Load of the Column

Figure 14 is the graph plotted for failure load against the b/t ratio for columns with different heights. It was noted that load at failure is increased with the increment of b/t ratio.

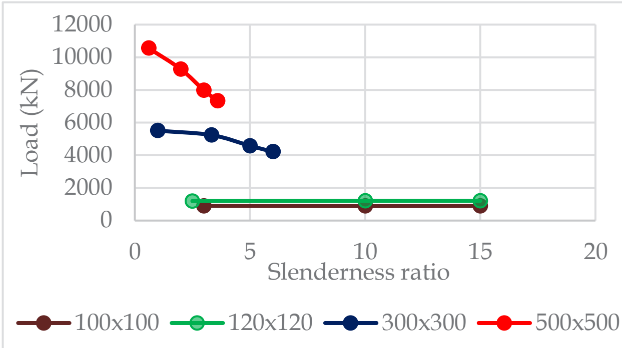


Figure 12: Effect of variation of column height on axial load carrying capacity of columns with slender c/s

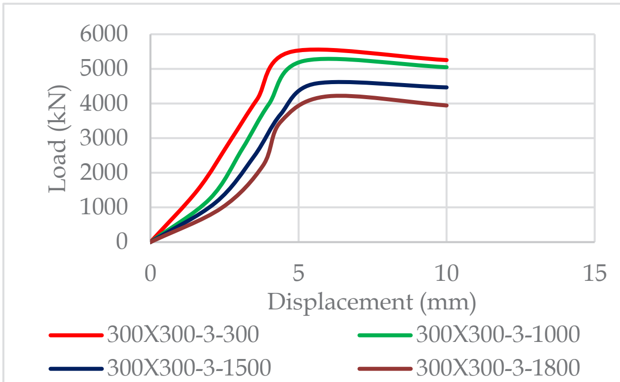


Figure 13: Effect of slenderness on the failure load of the column

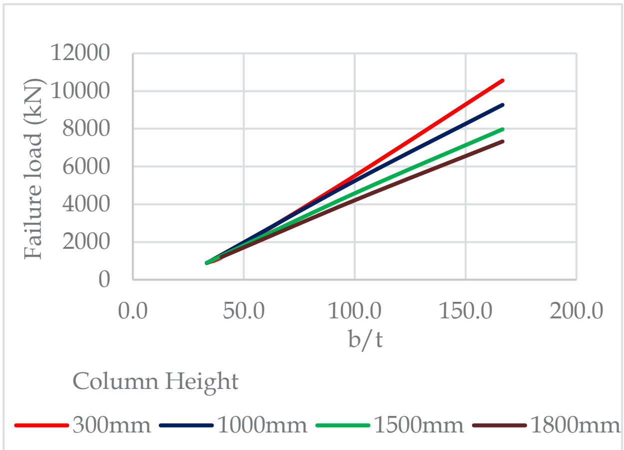


Figure 14: Effect of b/t ratio on failure load of column

6. Comparison with Code Provisions

The axial load carrying capacity of the parametric models was calculated according to the given provisions in BS 5400 – 5, Eurocode 4 and model defined by K. A.D.I. Bsisu [2]. Then the results were compared with numerical model results to identify the compatibility of the existing provisions on CFST columns to be used in the design of steel jacketed RC columns. Table 3 is a summary of the comparative analysis and for a detailed comparison, Figure 15 is provided.

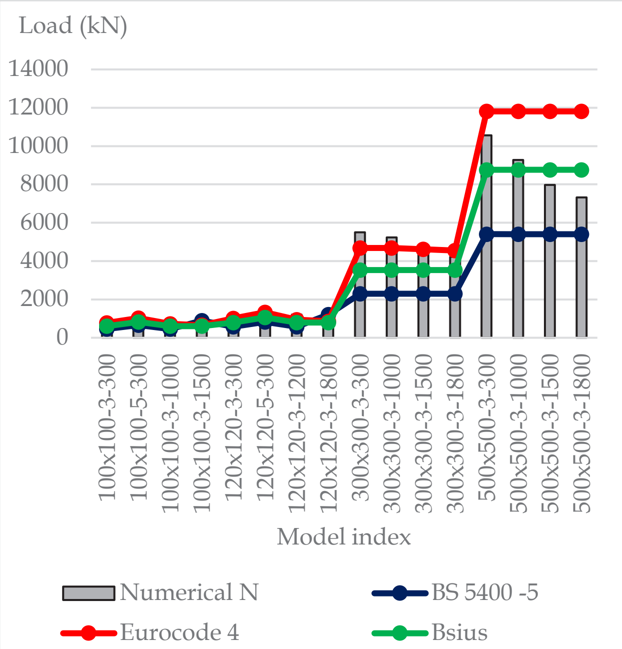


Figure 15: Code comparison with numerical model results

There it was noted that predictions by BS 5400 – 5 are almost half the results by the numerical model. Results by the model of Bsisu are constant for the same cross-section size, where it is only considering the contribution of cross-section area for load calculation but not the slenderness of the column. The Eurocode 4 is seeming to be more compatible to use with design of steel jacket RC columns as it shows higher similarity with the numerical model results in comparison to the other models. But to the end of the curve, for Eurocode 4 the results show no variation with the change of column height and those four column models were less than provided requirement of steel contribution ratio (0.2-0.9). Since the comparison shows a higher variation there, the provisions to be developed to be used with lesser steel contribution ratio. In general, all the mathematical models compared in this research are usable with columns with plastic cross-section size where b/t <40.

Since the predictions by Eurocode 4 are the most compatible with the numerical model results, a correlation factor (CF) was developed using Equation 13. Table 4 is a summary of the calculation of CF.

$$CF = \frac{Load\ predicton\ by\ numerical\ model}{Load\ prediction\ by\ Eurocode\ 4} \qquad \ldots(13)$$

Table 3: Comparison of numerical model results with predictions by design codes

Model index	Numerical Results (kN)	BS 5400-5 (kN)	Eurocode 4 (kN)	Model by Bsisu, 2002 (kN)
100x100-3-300	897.5	454.9	770.8	604.2
100x100-5-300	1144.6	662.4	1030.0	824.5
100x100-3-1000	882.8	454.2	720.7	604.2
100x100-3-1500	894.8	532.7	652.9	604.2
120x120-3-300	1194.7	582.3	1018.8	792.5
120x120-5-300	1496.5	828.1	1326.0	1053.7
120x120-3-1200	1204.8	581.2	943.4	792.5
120x120-3-1800	1205.8	681.5	852.7	792.5
300x300-3-300	5504	2298.3	4690.8	3528.2
300x300-3-1000	5235	2296.9	4690.8	3528.2
300x300-3-1500	4580	2295.9	4611.1	3528.2
300x300-3-1800	4210	2295.3	4554.8	3528.2
500x500-3-300	10562	5408.8	11810.8	8764.2
500x500-3-1000	9269	5406.9	11810.8	8764.2
500x500-3-1500	7975	5405.2	11810.8	8764.2
500x500-3-1800	7329	5404.3	11810.8	8764.2

Table 4: Correlation factor

Model index	Numerical model results (kN)	Eurocode 4 (kN)	Correlation factor
100x100-3-300	897.5	770.8	1.2
100x100-5-300	1144.6	1030.0	1.1
100x100-3-1000	882.8	720.7	1.2
100x100-3-1500	894.8	652.9	1.4
120x120-3-300	1194.7	1018.8	1.2
120x120-5-300	1496.5	1326.0	1.1
120x120-3-1200	1204.8	943.4	1.3
120x120-3-1800	1205.8	852.7	1.4
300x300-3-300	5504	4690.8	1.2
300x300-3-1000	5235	4690.8	1.1
300x300-3-1500	4580	4611.1	1.0
300x300-3-1800	4210	4554.8	0.9
500x500-3-300	10562	11810.8	0.9
500x500-3-1000	9269	11810.8	0.8
500x500-3-1500	7975	11810.8	0.7
500x500-3-1800	7329	11810.8	0.6

7. Conclusions

1. The capacity of the existing concrete column could be increased up to 60% by the introduction of the steel jacket as per the numerical models developed in this study.
2. Use of unconfined concrete properties for modelling was found to be acceptable since confinement effect is calculated by the Concrete Damaged Plasticity model of Abaqus.
3. A small increment of jacket thickness can cause nearly 25%-28% of capacity increment.
4. Capacity prediction by the numerical model was double the value of calculations by BS 5400-5.
5. The mathematical model proposed by Bisus is not applicable to all the column sizes
6. The predictions by EURO Code 4 resulted in a correlation factor of 1 to the nearest first decimal place. Therefore, Euro code 4 can be used to get more accurate design loads in comparison to other models
7. But the provisions to be developed for columns with steel contribution ratio less than 0.2 and to use with slender columns

References

[1] B. Kiriparan, W. J. B. S. Fernando, W. Bhathiya, P. Mendis, A. D. De Alwis, and H. Dilina, "Application of Steel-Concrete Composite Columns in Building Construction," vol. 1, pp. 1–8, 2018.

[2] K. A.-D. I. Bsisu, "Retrofitting of Square Reinforced Concrete Columns Subjected to Concentric Axial Loading with Steel Jackets," *CEO Al-Eman Hous. Co.*, 2002.

[3] H. S. Hadad, M. T. Nooman, M. Mohammed, and A. Mostafa, "The Effect of Confinement and Shear Connector on the Ultimate Capacity of the Short Composite Column," vol. 3, no. 4, pp. 190–204, 2015.

[4] Q. Zhang and M. Schäfer, "Comparison of Design for Composite Beams in Steel and Concrete According To Eurocode 4 and Chinese Design Codes," *Ce/Papers*, vol. 1, no. 4, pp. 426–437, 2017.

[5] F. Hafiz, "Analytical and Numerical Study of Concrete Filled Tabular Columns," *Int. J. Civ. Eng.*, vol. 3, no. 9, pp. 1–7, 2016.

[6] M. Abramski, "Load-carrying capacity of axially loaded concrete-filled steel tubular columns made of thin tubes," *Arch. Civ. Mech. Eng.*, vol. 18, no. 3, pp. 902–913, 2018.

[7] Amiya Kumar Samanta and Amit Paul, "Evaluation of Current Design Practices on Estimation of Axial Capacity of Concrete Encased Steel Composite Stub Columns: A Review," *J. Civ. Eng. Archit.*, vol. 7, no. 9, pp. 1080–1091, 2013.

[8] Q. Qiao, W. Zhang, Z. Qian, W. Cao, and W. Liu, "Experimental study on mechanical behavior of shear connectors of square concrete filled steel tube," *Appl. Sci.*, vol. 7, no. 8, 2017

[9] B. Uy, "Local and post-local buckling of concrete filled steel welded box columns," *J. Constr. Steel Res.*, vol. 47, no. 1–2, pp. 47–72, 1998.

[10] The European Union, *EN 1994-1-1 (2004) (English): Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings*. 2004.

[11] Y. R. Abbas, "Nonlinear Finite Element Analysis to the Circular CFST Stub Columns," *Procedia Eng.*, vol. 173, pp. 1692–1699, 2017.

[12] V. K. Papanikolaou and A. J. Kappos, "Confinement-sensitive plasticity constitutive model for concrete in triaxial compression," *Int. J. Solids Struct.*, vol. 44, no. 21, pp. 7021–7048, 2007.

[13] Z. Tao, Z. Bin Wang, and Q. Yu, "Finite element modelling of concrete-filled steel stub columns under axial compression," *J. Constr. Steel Res.*, 2013.

[14] M. F. Ferrotto, L. Cavaleri, and F. Di Trapani, "FE modeling of Partially Steel-Jacketed (PSJ) RC columns using CDP model," *Comput. Concr.*, vol. 22, no. 2, pp. 143–152, 2018.

[15] L. H. Han, G. H. Yao, and Z. Tao, "Performance of concrete-filled thin-walled steel tubes under pure torsion," *Thin-Walled Struct.*, vol. 45, no. 1, pp. 24–36, 2007.

[16] K. A. S. Susantha, H. Ge, and T. Usami, "A capacity prediction procedure for concrete-filled steel columns," *J. Earthq. Eng.*, vol. 5, no. 4, pp. 483–520, 2001.

[17] T. W. Kirkbridge, "BS 8110 Structural Use of Concrete.," *Precast Concr*, vol. 4, no. 1, pp. 31–32, 1973.

[18] B. Alfarah, F. López-Almansa, and S. Oller, "New methodology for calculating damage variables evolution in Plastic Damage Model for RC structures," *Eng. Struct.*, vol. 132, no. February, pp. 70–86, 2017.