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Ultimate Strength of RC Chimney Section under Monotonic Loading

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Abstract: The dispersion of particulate air pollutants and impure gaseous residue through tall stack-like structures, including chimneys, has played an important role in industrial air pollution. Tall RC chimneys can be regarded as cantilever columns subjected to (i) axial load resulting from the weight of the shell, lining, and other components and (ii) bending moment resulting from lateral loads involving wind and earthquake forces. The 3rd edition of IS: 4998 published in 2015, adopted the limit state method for the design of RC chimneys. This methodology requires a clearly defined and precise stress-strain relationship for concrete and steel, yet many discrepancies are evident when the stress-strain relationships of these materials adopted by IS: 4998-2015 and other well-designed standards are compared. The paper also discusses the effects of these disparities of material law on the ultimate strength of the section, along with the contribution of concrete and steel in ultimate strength, ultimate curvature, and depth of the neutral axis. For the comparison of the above-mentioned parameters, design recommendations of IS 4998 – 2015, CICIND 2011, and ACI 307 – 08 are used. The results of the study reveal that a higher percentage of steel the disparities are governed by the stress-strain law of steel rather than concrete. As for analysis and design purposes, RC chimneys are considered to be tall cantilever columns having thin-walled hollow circular sections subjected to axial compression resulted from self-weight and bending moments resulted from lateral forces. The results of this study can also apply to any such structures like TV towers.

Keywords: ACI: 307 – 08, CICIND 2011, IS: 4998 – 2015, Ultimate Strength, RC chimney.

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1. Introduction

The rigorous and strict regulations enforced by the environmental board committee have emerged the need to construct tall RC chimneys with a considerable height which will, in turn, control the increasing pollution caused by various gaseous and particulate pollutants.

RC chimneys are regarded as thin-walled hollow circular cantilever columns subjected to axial compression as well as bending moment. The self-weight of the chimney induces the axial load and the lateral forces induce the bending moment for the chimney. The wind force is considered a predominant lateral load in the design of the chimney as it is generally more critical than seismic force. The term 'ultimate strength' for an RC chimney can be defined as its ultimate moment carrying capacity when subjected to an axial compressive load associated with a particular neutral axis depth within the section. A very clear and appropriately specified stress-strain relationship between concrete and steel is required for the analysis of chimneys as per the limit state condition. Different codes have defined different stress-strain relationships between concrete and steel for this purpose.

Limit state approach for the design of RC chimneys has been adopted in the latest revision of IS: 4998-2015. Much analytical research work has long been carried out before the introduction of this approach in the code. Rai et al. (2006) utilized the stress-strain curve specified in IS: 456-2000 and plotted load-moment interaction curves for hollow tubular sections. The comparison of these curves with the then-existing interaction curves specified in IS: 11628-1985 and IS: 4998-1975 concluded that the limit state method shall be adopted for such hollow tubular sections rather than the working stress method which was then being implemented. Babu and Yaragal (2007) also plotted the load-moment interaction diagram for which they developed a computer program using a simplified rectangular stress block for concrete. Even though this research was useful to a certain extent, neither the use of a rectangular concrete stress-block nor the stress-strain relationship specified in IS: 456-2000 can be fully justified for thin-walled hollow tubular sections like that of a chimney. Rao and Menon (1995) then introduced and suggested a completely new stress-strain relationship of concrete for such hollow circular sections and compared the plotted interaction curves with those of various other well-established codes, namely CICIND 1984, Pinfold (1984) and DIN 1056 – 1984. The present paper focuses on the detailed comparison of IS: 4998-2015 with CICIND 2011 and ACI 307 – 08.

2. Research Significance

Tables 1 and 2 clearly manifest the dissimilarities prevalent in the various design provisions with regards to specified load, strength reduction factors, maximum strain limits for concrete as well as steel, and the criteria for estimating the ultimate strength of hollow circular RC thin-walled sections.

Table 1. Disparities in strength reduction factors

Design Standards	Strength Reduction Factors		
	Concrete (γ_c)	Steel (γ_s)	Overall
IS: 4998 – 2015	1.5	1.15	-
CICIND 2011	1.5	1.15	-
ACI 307 – 08	-	-	0.8

Table 2. Disparities in strain limits and elastic modulus

Design Standards	Strain Limits		Modulus of Elasticity of Steel (E_s)
	Concrete ($\varepsilon_{c,\ max}$)	Steel ($\varepsilon_{s,\ max}$)	
IS: 4998 – 2015	0.002	0.05	2×10^5
CICIND 2011	0.003	0.01	2.1×10^5
ACI 307 – 08	0.003	0.07	2×10^5

Given the numerous discrepancies in the stress-strain relationships in the stated well-established design standards, the present study pertaining to the analysis and analogy of the well-established international design codes with that of IS: 4998 – 2015 shall considerably enhance the knowledge in this aspect.

The scope of the paper is limited to the analytical study based on the ultimate strength of the RC chimney section carried out through specially designed Spreadsheet programs as per guidelines of IS: 4998 – 2015, CICIND 2011, and ACI 307 – 08 and further, the comparison of its results by the contribution of its constituent materials (concrete and steel), ultimate curvature and the depth of neutral axis.

3. Material Law

The disparities in the stress-strain relationship of concrete and steel adopted by the three design standards, i.e., IS: 4998 – 2015, ACI 307 – 08, and CICIND 2011 are discussed here in detail along with the bases for the same in the design standards.

3.1. Stress-Strain Curve for Concrete

In the 3rd revision of IS: 4998, i.e., in IS: 4998 – 2015, it is assumed that the concrete compressive stress is increasing parabolically from zero at the neutral axis location to the peak value of 0.002 as indicated in Fig. 1. When compared to the other two codes, it is found that the IS: 4998 – 2015 introduces the short-term loading factor and specifies the stringent value of strain in concrete at the center of the thickness of shell at failure as 0.002. This effect of short-term loading was specifically presented in the CICIND 1984 edition, in which parabolic stress-strain relation is assumed for concrete when the section is analyzed under long-term loading, and linear stress-strain relation for the same under short-term loading. The compressive strain in concrete at failure was assumed to be 0.001, which is based on the research work of Schueller and Bucher (1997) which concludes that the fracture is not caused by repeated oscillation loading due to wind but by a single wind gust in reinforced concrete chimneys.

The above-mentioned material law for concrete was adopted from the results of the experimental research work carried out by Naokowski (1981). The reinforced concrete test specimens were tested under reversed cyclic loading by him for the study. However, the results of this experimental research work based on reverse cyclic loading do not apply to the typical along wind conditions considered in the CICIND 1984 code, as the behavior of along-wind loading is better approximated by monotonic loading rather than reverse cyclic loading as the along-wind loading and its response is somewhat quasi-static. However, in the newest edition of the CICIND Code, i.e., CICIND 2011, this short-term loading effect is ignored and the parabolic stress-strain relationship for concrete in flexural compression under permanent loading is considered and the concrete strain at the center of the thickness at failure is limited to 0.003 as shown in Fig. 2.

On the other hand, IS: 4998 – 2015 brings in the short-term wind loading effect. To consider this effect, the short-term loading factor (C_{sf}) is presented which was first introduced by Rao and Menon (1995). This C_{sf} factor is influenced by the amount of axial compression (P_u) on the RC chimney section. The C_{sf} factor varies from 1.12 to 1 for $P_u = 0$ to $P_u = P_{u,\ max}$ i.e. under pure compression respectively. In their research paper, Rao and Menon (1995) established the foundation of a new model of the stress-strain curve of concrete for RC chimney sections which is currently adopted in the newly revised edition of IS: 4998 – 2015. Further expanding the research works of Rusch (1960) and Ellingwood et al. (1980), a logical formwork for the concrete stress-strain relationship has been proposed by Rao and Menon (1995), which accounts for the effects of tubular geometry and short-term loading effect in the form of short-term loading factor C_{sf}.

Based on a large number of experiments carried out on eccentrically loaded concrete tubular sections under varying load durations, a family of stress-strain curves was presented by Ruch (1960). The observations of these tests led to the conclusion that if the maximum concrete strain is limited to 0.002 then it is reasonable to assume an increase of approximately 10% in stress for relatively short-term loading. In their reliability study of reinforced concrete columns, Ellingwood et al. (1980) had also recommended a similar strength improvement factor; under wind loading conditions. The short-term loading factor (C_{sf}) can be calculated using Eq. 1 and Eq. 2 as per the IS: 4998 – 2015.

C_{sf} = [0.95 – 0.1 (Pu / Pu, max)] / 0.85
(1)

Where,

Pu = Factored axial load and

$$P\left[\left(\frac{0.67\ ck}{\gamma_c}\right)\left(1-\frac{p_t}{100}\right)+f_s(\varepsilon_{cu})\left(\frac{p_t}{100}\right)\right]_{umax} \quad (2)$$

As the diameter of the RC chimney is very large compared to its thickness, the strain gradient across the thickness at the extreme compression location is minimal and hence, the stringent value of 0.002 for maximum strain in concrete in the axial compression seems to be valid even in the presence of bending.

However, ACI: 307 – 08 follows Whitney's rectangular stress block as prescribed in ACI: 318 – 2002. This stress block is proved to be valid for rectangular and flanged sections but it cannot be directly used without modification for the RC chimney section (thin-walled hollow circular section) in which the maximum compressive strain in concrete is less than 0.003 when the fracture limit of steel is reached, i.e., the compressive stress block is not fully developed and hence, some modifications as shown in Fig. 3 are adopted in ACI: 307 – 08. The εc, max in compression is assumed as 0.003 and to include the above consequences, the modification factor (Q) is introduced in the stress block of concrete which is based on the experimental work of Morkin and Rumman (1985) and the numerical study carried out by the committee.

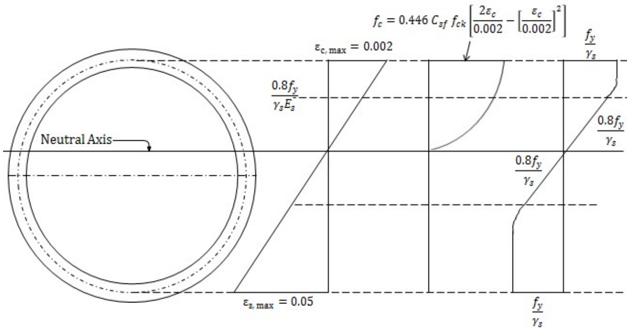

Fig. 1. Stress-strain curve for concrete and steel as per IS: 4998 - 2015

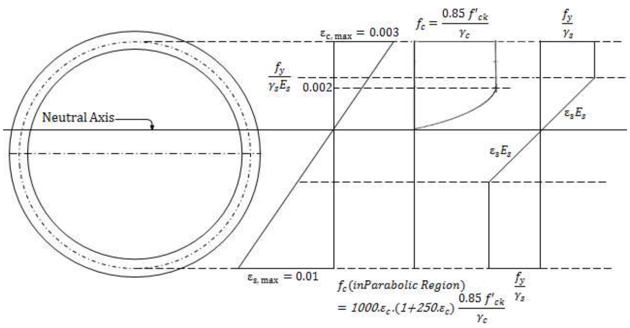

Fig. 2. Stress-strain curve for concrete and steel as per CICIND 2011

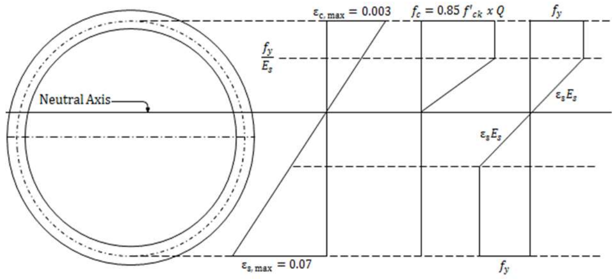

Fig. 3. Stress-strain curve for concrete and steel as per ACI: 307 – 08

3.2. Stress-Strain Curve for Steel

The stress-strain relationship adopted by the three design standards is shown in Figs. 1, 2, and 3. The idealized elastoplastic stress-strain relationship for steel in tension and compression is adopted by ACI: 307 – 08 and CICIND 2011, which assumes that steel, shows sharp yielding behavior. IS: 4998 – 2015 on the other hand, has incorporated the offset approach to determine the yield strength. In this approach, the characteristic strength of the steel is taken as that stress at which the steel shows definite limiting deviation from stress-strain proportionality, the value of which is taken as 0.2% of proof stress. Paulson et al. (2016) have compared the ultimate strength of reinforced concrete column sections and beam sections using different stress-strain relationships of steel, i.e., for sharp yield plateau and for gradually yielding curves wherein the offset values are taken as 0.1% and 0.2%, and concluded that the approach of using 0.2% offset is safe and practical.

4. Formulation of Spreadsheet Program

The ultimate moment capacity (M_u) is the moment when the strain in concrete reaches its maximum value. The (P_u) and (M_u) corresponding to any given normal compression are determined by solving the equilibrium Eq. 3 and 4:

$$P_u = P_{uc} + P_{us} \quad (3)$$

$$M_u = M_{uc} + M_{us} \quad (4)$$

Where (P_{uc}) and (P_{us}) are the resultant ultimate forces obtained from concrete and steel stress blocks, and (M_{uc}) and (M_{us}) denote the moments about the centerline of that tubular section.

To calculate the ultimate moment capacity of the RC chimney section, a generalized spreadsheet program is developed using MS Excel. The inputs of this program are material properties like concrete characteristic compressive cube strength (f_{ck}) as per IS: 456 – 2000, characteristic cylindrical compressive strength ($f'_{ck} = 0.8\ f_{ck}$) as per CICIND 2011 and ACI: 307 – 08, steel yield strength (f_y), the elastic modulus of concrete (E_c) and elastic modulus of steel (E_s) and geometrical properties of the section like outer diameter (D), thickness (t), location of steel bars and percentage of steel (p_t). The axial force is also an indirect input, which is obtained by adjusting the depth of the neutral axis (X_u).

A standard chimney cross-section consisting of outer diameter 10 m, thickness 0.45 m, concrete strength f_{ck} = 40 MPa and reinforcement yield strength fy = 500 MPa are used in this study together with the percentage of reinforcement as pt = 0.3%, 0.5%, 0.75%, 1.0%, 1.5% and 2.0% and axial stress ratio n = 0 to 0.2 at increments of 0.025. Where (f_c) is the axial stress generated due to the axial force on the section. The general procedure followed to obtain ultimate strengths is as follows:

Step 1: The cross-section is entirely divided into several small strips parallel to the neutral axis. To maintain the accuracy of the results, the width of each strip is maintained as 1 mm.

Step 2: Then considering the assumption that the plane section remains plane before and after bending, the value of strain at the geometrical center of each strip and the location of reinforcement are calculated separately.

Step 3: After this, concrete stress at the geometrical center of each strip and the steel stress at the location of reinforcement are calculated individually for IS: 4998 – 2015, CICIND 2011, and ACI: 307 – 08. It is to be noted here that the stresses for ultimate conditions for these design standards are calculated as per their design recommendations shown in the form of stress blocks in Fig. 1, 2, and 3.

Step 4: The concrete stresses at different levels are then multiplied with an area of the respective strip to get concrete compressive force and the steel stresses at different locations are multiplied with the associated area of reinforcement to calculate the net force offered by reinforcement i.e. the compressive force minus the tensile force obtained at either side of the neutral axis. Then to get the resultant force (P_u) the concrete compressive force and net force of steel are algebraically added.

Step 5: The depth of the neutral axis is then adjusted using the 'Goal Seek' function of the MS Excel so that the obtained Pu from the above steps could be matched with the P_u which is an input in the form of axial stress ratio f_c/f_{ck} as mentioned above.

Step 6: Then these concrete and steel forces are multiplied with their centroidal distances separately to get the moment from concrete and steel, respectively. These moments are then algebraically added to get the resultant moment (M_u).

5. Results and Discussion

The results of the comparative study between the three design codes, carried out on reinforced concrete chimney sections for the percentage of reinforcement p_t = 0.3%, 0.5%, 0.75%, 1.0%, 1.5%, and 2.0%, and axial stress ratio n = 0 to 0.2 are discussed in this section in terms of ultimate moments along with the contribution of its constituent materials namely concrete and steel, ultimate curvatures and depth of neutral axis.

5.1. Ultimate Moments

The ultimate moment capacity (M_u) of the section calculated for three design methods is compared in this section along with the comparison of the contribution of the constituent materials, namely concrete and steel. The salient observations from the above graphs are listed below:

1. The disparities among the various methods are small when (n) is low and the reinforcement ratio is minimum. However, they become increasingly pronounced with an increase in n and tend to be very large at high reinforcement ratios. When the axial stress ratio (n) is low and the section is lightly reinforced, extreme tension condition prevails, wherein the concrete stress-strain curve has hardly any contribution to make. However, as n increases, the section is subjected to more compression, and hence the difference between the various methods gets emphasized. Fig. 8 compares the values of the depth of the neutral axis (X_u) and also confirms the above statement.
2. Fig. 4 also indicates an increase in moment strength with an increase in axial load and reinforcement ratio for 'ACI' and 'CICIND', but on the other hand at higher ratios, it can be observed that the ultimate strength obtained from 'IS' which is highest amongst the three codes up to lower stress ratio, has substantially reduced non-linearly and falls below the values obtained from 'ACI' and 'CICIND'; this reduction is significant for higher steel ratios i.e. for 1.5% and 2%. To understand the difference between values of the ultimate moment contribution of concrete and steel, the graphs are separately plotted.

It can be seen from Fig. 5, that the share of ultimate capacity of concrete increases with an increase in stress ratio for all three methods. However, it can be observed that the share of concrete calculated using the 'IS' methodologies is higher than the share of concrete obtained from the 'ACI' and 'CICIND' methodologies. This increment in the share of concrete obtained using the 'IS' stress-strain relationship is more significant at a higher stress ratio and a lower percentage of steel.

It is also observed from Fig. 6 that at a higher stress ratio, the contribution of steel in ultimate moment capacity is reduced when calculated using the 'IS' methodology as compared to the methodologies using the other two codes, and hence, this decrement creates a difference in the results of Indian design code with the other design codes. Also, this difference further increases with an increase in the percentage of steel.

From Figs. 4, 5, and 6, it can be concluded that at a lower percentage of steel and up to a stress ratio of 0.15, the values of (M_u) are higher for 'IS' compared to 'ACI' and 'CICIND' as the share of concrete is higher in that range.

Beyond the stress ratio of 0.15 the reduction in (M_u) calculated from 'IS' is mainly due to the reduction in the contribution of steel in (M_u) which is significant at a high percentage of steel, as the share of concrete is still increasing but this share is having only a marginal increase in comparison with 'ACI' and 'CICIND' at a higher percentage of steel.

5.2. Ultimate Curvature

The ultimate curvature (ϕ_u) is calculated by dividing the nominal ultimate concrete strain by the depth of the neutral axis. The results plotted in Fig. 7 indicate the reduction in curvature capacity as the axial stress ratio or the reinforcement ratio increases. The comparison indicated a reasonable correlation although the 'IS' design methodologies slightly underestimated the ultimate curvature values. The differences could be attributed to the neutral axis depth being greater for 'IS'. The differences can be expected to be greater when the axial load and neutral axis depths increase.

It can also be seen from Fig. 5 that the RC chimney sections designed using codal provision given in the CICIND 2011 give higher ductility in all the cases. The values of ductility for the particular percentage of steel decrease with an increase in the axial stress ratios.

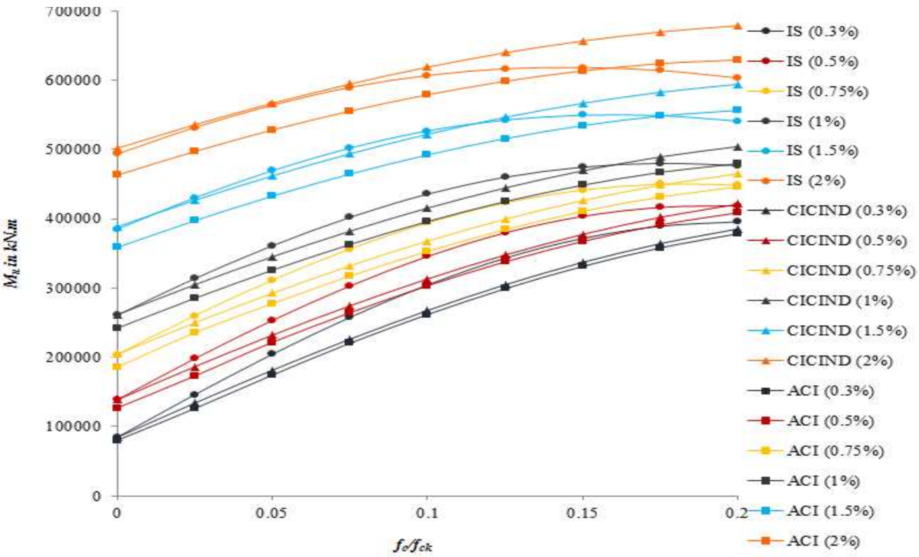

Fig. 4. Comparisons of ultimate moments

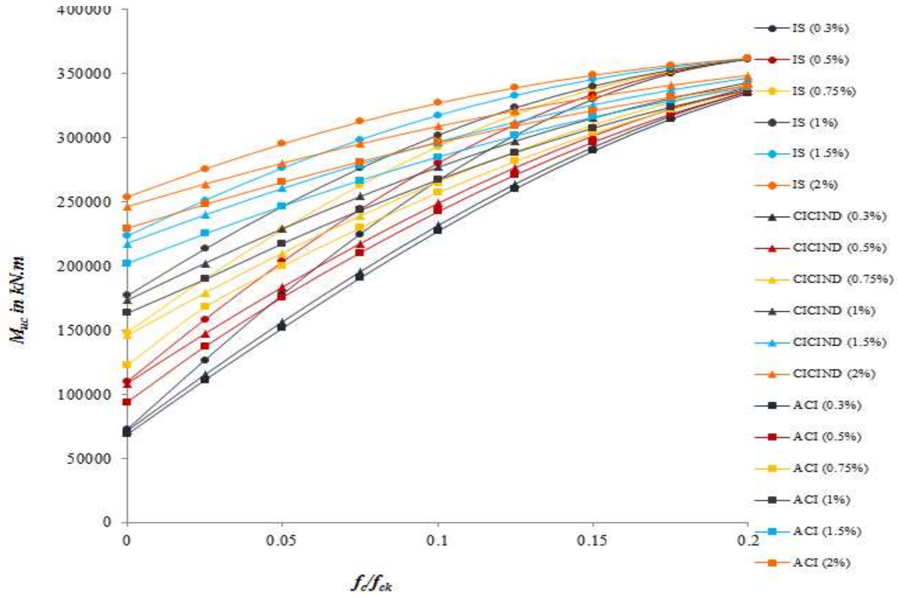

Fig. 5. Comparisons of the contribution of concrete in ultimate moments

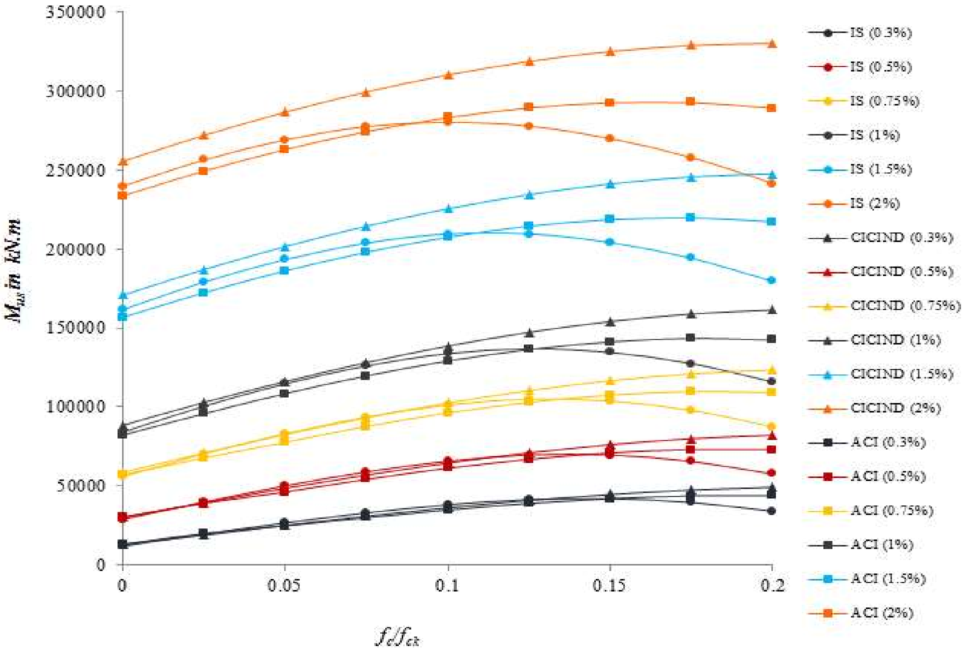

Fig. 6. Comparisons of the contribution of steel in ultimate moments

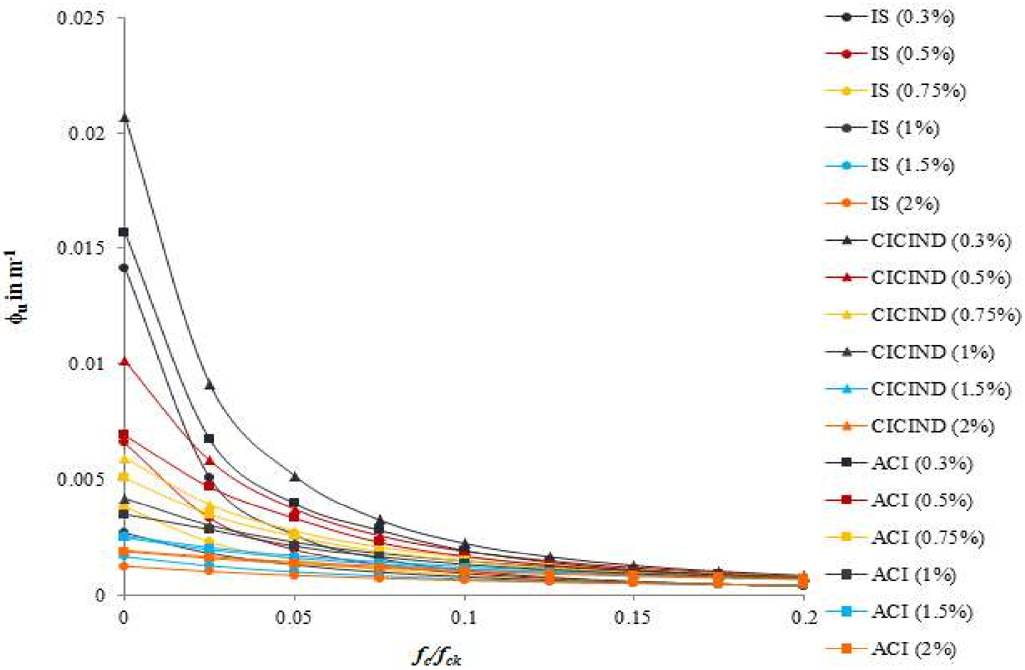

Fig. 7. Comparisons of ultimate curvature

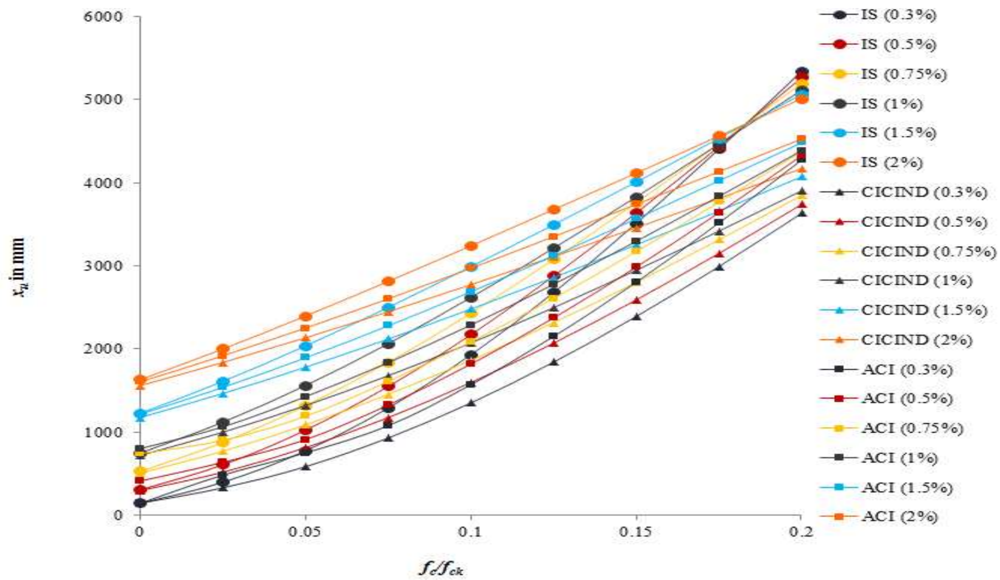

Fig. 8. Comparisons of the depth of neutral axis

5.3. Depth of Neutral Axis

The neutral axis depth measured from the centreline of the reinforced concrete chimney section associated with the ultimate moment is calculated using the generalized computer program. The neutral axis depth increases with an increase in the axial stress ratio and percentage of steel. It varies from 141 mm to 1629 mm for 0.3% reinforcement and 5339 mm to 5010mm for 2% of reinforcement. It can also be seen that differences between the three codes are almost negligible at zero stress ratio and increase at higher stress ratio.

6. Conclusion

Analytical strengths of hollow thin-walled tubular reinforced concrete chimney sections incorporating the stress-strain models of concrete and steel given in IS: 4998 – 2015, ACI 307 – 08, and CICIND 2011 are compared.

There exist considerable differences between the above three prevailing methods to estimate the ultimate moment capacity of the tubular reinforced concrete section, subject to wind moment combined with normal compressive dead load. These disparities are predominant at higher stress ratio and high steel ratio. These disparities are due to differences in the various stress-strain models of concrete at a lower percentage of steel, i.e., up to 0.75% and at a lower stress ratio, i.e., up to 0.125; but at a higher percentage of steel, i.e., from 1% to 2%, these disparities are due to the difference in stress-strain models of steel adopted by the three codes.

The strain specified in the distribution as per 'ACI' and 'CICIND' (0.003) methods are larger than those specified by the 'IS' method (0.002); hence the corresponding steel stresses are also larger, both in compression and tension. But this is offset by the descending branch of the concrete stress-strain curve of the 'ACI' method, which results in a smaller concrete stress block area. At lower values of steel percentage, this influence of concrete is overriding; also, at higher values of steel percentage, the increased influence of steel results in 'ACI' and 'CICIND' predicting larger strengths than 'IS', as in 'IS,' none of the steel in the tension zone reaches the yield strength whereas, in the 'ACI' and 'CICIND', most of the steel in the tension zone acquires the full yield strength.

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