



Research Article

Moment-curvature relationships for RC chimney sections

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Abstract

The 3rd revision of IS: 4998 i.e. 2015 edition adopted a limit state method of design for RC chimney and gave a new model of stress-strain relationship for concrete and steel. RC chimneys are tall cantilever columns having thin-walled hollow circular sections and subjected to axial compression resulting from self-weight and bending moment resulting from lateral forces. It has been seen that there are many disparities between the stress-strain relationship of these materials adopted by IS: 4998 – 2015 and other well-established design standards. This paper compares and discusses these disparities in terms of strength, ductility factor and energy absorption with the help of plotted moment-curvature charts. For the comparison, design recommendations of IS: 4998 – 2015, CICIND 2011, ACI: 307 – 08 are used. Moment-curvature curves are plotted using the major two conditions of the section i.e. yielding of tension reinforcement and crushing of concrete in compression. It is obtained from the study that CICIND 2011 gives higher results for all parameters, namely strength, ductility factor and energy absorption.

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1. Introduction

In the 3rd revision of IS: 4998 proposed in 2015 [1], limit state design approach has been adopted to design a section of reinforced concrete (RC) chimney. Many researchers have contributed to this field before this revision took place. The load-moment (P_u-M_u) interaction curves for the tubular sections like chimney were plotted and compared by Durgesh C. Rai and others in 2006 [2] using limit state theory according to IS: 456 – 2000 [3] provisions and also using working stress theory according to the provisions of IS: 11628 – 1985 [4] and IS: 4998 – 1975 [5]. The comparison showed that it would be preferable to use limit state design approach rather than using the working stress method for the tubular section like RC chimneys, TV towers, etc.

To plot load-moment (P_u-M_u) diagrams, a computer program has been developed by K. S. Babu Narayan and Subhash C. Yaragal in 2007 [6]. It is to be noted here that the computer program uses simplified rectangular stress-strain curve to plot the load-moment interaction curve of hollow circular section. But behaviour of thin-walled hollow circular sections under the action of axial compressive stress and flexure is different than the regular solid beam or column section and hence, for such tubular thin-walled sections the direct use of stress-strain curve for concrete specified in IS: 456 – 2000 as well as rectangular stress block should be avoided.

A new model that represents the stress-strain relationship of concrete for thin-walled hollow circular sections has been proposed by P. Srinivasa Rao and Devdas Menon in 1995 [7]. Using this proposed new model of stress-strain relationship for concrete, the P_u-M_u interaction curves were plotted for the tubular sections and the same were compared with interaction curves plotted using the criteria of various other well-

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established codes namely ACI: 307 – 88 [8], CICIND 1984 [9], DIN 1056 – 1984 [10] as well as using the criteria given in the book of Pinfold 1984 [11]. This new model of stress-strain curve for concrete has been adopted for the limit state design approach in IS: 4998 – 2015. A comparison of various prevalent design provisions with regards to strength reduction factors, maximum strain limits for concrete and steel, and modulus of elasticity of steel for RC chimney sections are shown in Table 1 that reveal significant disparities. Thus, the comparison of these codes becomes necessary to enhance the knowledge base on this topic.

Table 1. Design criteria in different design codes

Design Standard	Load Factor		Strength Reduction Factor			Strain Limit		Elastic Modulus (MPa)
	Dead Load	Wind Load*	Concrete (γ_c)	Steel (γ_s)	Overall	Concrete ($\epsilon_{c, max}$)	Steel ($\epsilon_{s, max}$)	Steel (E_s)
IS: 4998 – 2015	0.9 or 1.2	1.6	1.5	1.15	-	0.002	0.05	2×10^5
CICIND 2011	1.0	1.6	1.5	1.15	-	0.003	0.01	2.1×10^5
ACI: 307 – 08	0.9 or 1.2	1.6	-	-	0.8	0.003	0.07	2×10^5

* Factor 1.6 shall be used for Along Wind Load

To carry out non-linear analysis of reinforced concrete elements, moment-curvature relationship is widely used [12]. The bilinear law having first division going from the zero-load condition up to the point representing the yielding of tension reinforcement, and the second division going from this yield point to the point representing the crushing of concrete in compression is the modest yet satisfactorily correct description of moment-curvature ($M-\phi$) relationship for RC sections. In this paper, moment-curvature curves are plotted to compare the values of ultimate strength, ductility and energy absorption obtained from the design approaches stated in IS: 4998 – 2015 [1], CICIND 2011 [13] and ACI: 307 – 08 [14].

2. Comparison of Stress-Strain Relationship for Materials

In this section the relationship between stress and strain for concrete and steel recommended by IS: 4998 – 2015 [1], CICIND 2011 [13] and ACI: 307 – 08 [14] for yield and ultimate conditions are compared in detail.

2.1 Relationship between Stress and Strain 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 neutral axis location to the peak value at the strain of 0.002 as indicated in (Fig. 1(a)). When compared to the other two codes, it is found that 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 [9] 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, G.I. and Bucher C.G. in 1983 [15] 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 in 1981 [16]. 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 behaviour 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. 1(b)).

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 P. Srinivasa Rao and Devdas Menon in 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 his research paper P. Srinivasa Rao and Devdas Menon in 1995 [7], established the foundation of a new model of 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 in 1960 [17] and Ellingwood in 1980 [18], a logical formwork for the concrete stress-strain relationship has been proposed by P. Srinivasa Rao and Devdas Menon [7], which accounts for the effects of tubular geometry and short-term loading effect in the form of short-term loading factor.

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 Rusch in 1960 [17]. The observations of these tests led towards 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 in 1980, Ellingwood and others [18] had also recommended a similar strength improvement factor; under wind loading conditions. The short-term loading factor, C_{sf} can be calculated using the equation given in the IS: 4998 – 2015 as follows:

$$C_{sf} = [0.95 - 0.1 (P_u/P_{u, max})] / 0.85 \quad (1)$$

where; P_u = Factored axial load and

$$P_{u, max} = 2\pi r t \left[\left(\frac{0.67 f_{ck}}{\gamma_c} \right) \left(1 - \frac{P_t}{100} \right) + f_s (\epsilon_{cu}) \left(\frac{P_t}{100} \right) \right] \quad (2)$$

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 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 [19]. 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 modification as shown in (Fig. 1(c))

are adopted in ACI: 307 – 08. The $\varepsilon_{c, max}$ 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 in 1985 [20] and the numerical study carried out by the committee.

Where, Q can be derived by the following equations:

For $\alpha \leq 5$ degrees

$$Q = (-0.523 + 0.181\alpha - 0.0154\alpha^2) + (41.3 - 13.2\alpha + 1.32\alpha^2) (t/r) \quad (3)$$

For 5 degrees $< \alpha \leq 10$ degrees

$$Q = (-0.523 + 0.181\alpha - 0.0154\alpha^2) + (41.3 - 13.2\alpha + 1.32\alpha^2) (t/r) \quad (4)$$

For 10 degrees $< \alpha \leq 17$ degrees

$$Q = (-0.488 + 0.076\alpha) + (9.758 - 0.640\alpha) (t/r) \quad (5)$$

For 17 degrees $< \alpha \leq 25$ degrees

$$Q = (-1.345 + 0.2018\alpha + 0.004434\alpha^2) + (15.83 - 1.676\alpha + 0.03994\alpha^2) (t/r) \quad (6)$$

For 25 degrees $< \alpha \leq 35$ degrees

$$Q = (0.993 - 0.00258\alpha) + (-3.27 + 0.0862\alpha) (t/r) \quad (7)$$

For $\alpha > 35$ degrees

$$Q = 0.89 \quad (8)$$

Where α is the one-half of the central angle subtended by neutral axis on the chimney section

2.2 Relationship Between Stress and Strain for Steel

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 behaviour. IS: 4998 – 2015 on the other hand, has incorporated the offset approach to determine the yield strength. In this approach, the characteristic strength of steel is taken as that stress at which it shows definite limiting deviation from stress-strain proportionality, the value of which is taken as 0.2% of proof stress. Conrad Paulson and others in 2016 [21] have compared the ultimate strength of reinforced concrete column 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.

Stress-strain relationship for concrete and steel for yield conditions are shown in (Fig. 2)

3. Methodology

Curves for moment-curvature ($M-\phi$) relationship for the above-mentioned design standards are plotted using four important points as follows:

Point 1: This point is start of loading. At zero loading there is zero moment and hence zero curvature i.e. the coordinates for this point are (0, 0).

Point 2: Since the concrete is not crushed and the steel is not yielded, the elastic moment capacity of the full section is used. This point is calculated from the linear stress-strain relationship of steel and concrete.

Point 3: The yield characteristics of the section are calculated by using the yield strain of steel in tension. For the most accurate solution, this point is obtained using the actual stress-strain relationship of steel and concrete given in the three design standards.

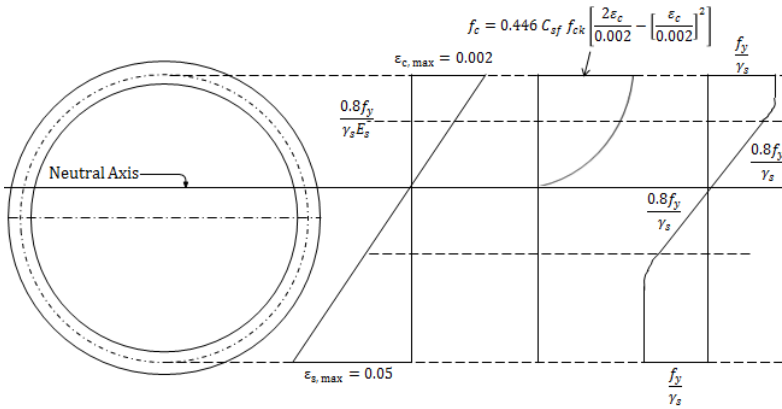
Point 4: The ultimate characteristics of the section are calculated by limiting the strain of concrete in compression at which it crushes. This point is obtained using the actual stress and strain relationship of steel and concrete as presented in (Fig. 1).

The above four points of the moment-curvature relationship curve generate three important regions of the curve.

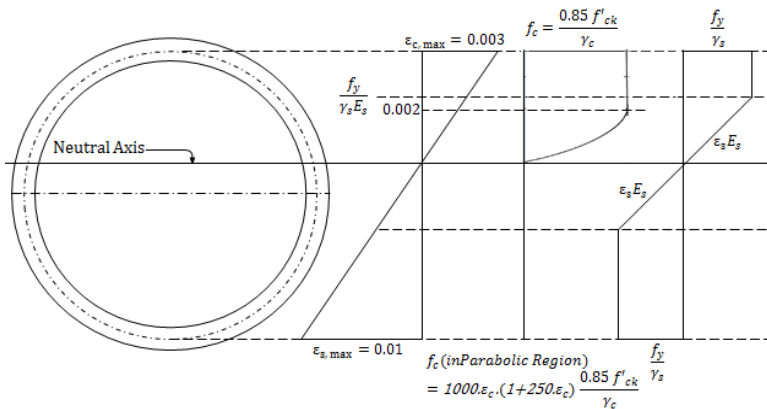
Region 1: between points 1 and 2. This region is called the uncracked response region.

Region 2: between points 2 and 3. This region is called the cracked linear response region.

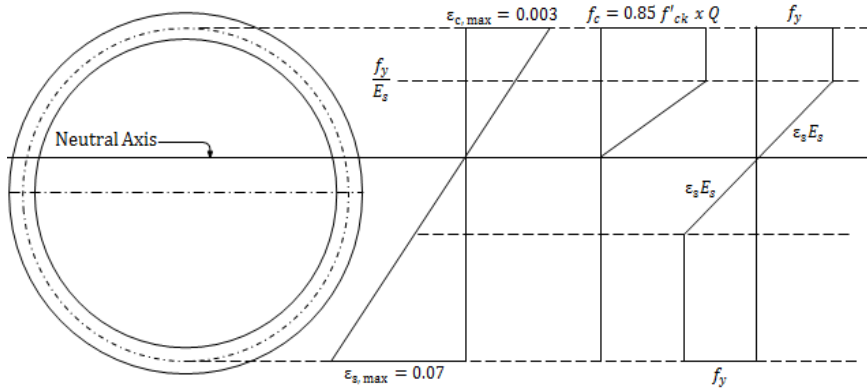
Region 3: between points 3 and 4. This region is called the inelastic response region.



(a)

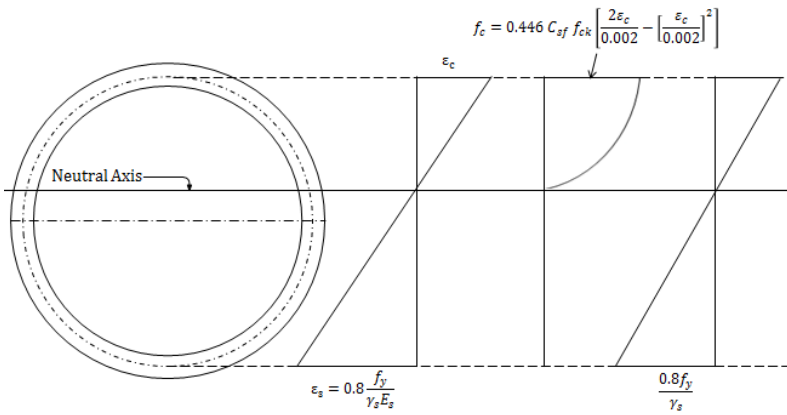


(b)

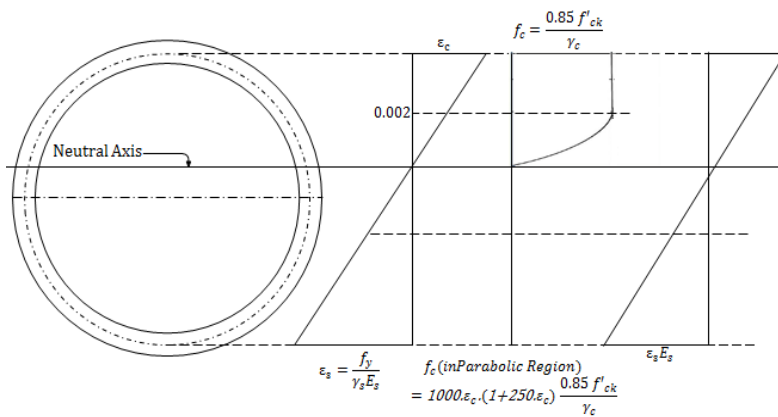


(c)

Fig. 1 Stress-strain relationship for concrete and steel for ultimate conditions as per (a) IS: 4998 - 2015 (b) CICIND 2011 (c) ACI: 307 - 08 (a) IS: 4998 - 2015 (b) CICIND 2011 (c) ACI: 307 - 08



(a)



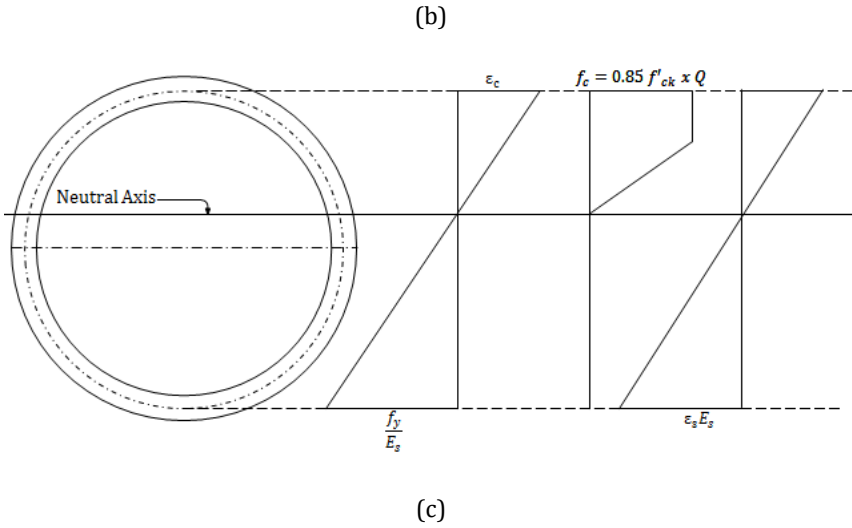


Fig. 2 Stress-strain relationship for concrete and steel for yield conditions as per (a) IS: 4998 - 2015 (b) CICIND 2011 (c) ACI: 307 - 08

4. Development of Spreadsheet Program

To calculate the x and y coordinates of each point of this moment-curvature relationship curve of 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), 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).

For the comparative study, a typical RC chimney cross-section comprising of $D = 10$ m, and $t = 0.45$ m, $f_{ck} = 40$ MPa and $f_y = 500$ MPa is selected with variation in $p_t = 0.5\%$, 0.75% , 1.5% and 2.0% and variation in $(n = f_{ca}/f_{ck}) = 0.025$, 0.05 , 0.1 , 0.15 and 0.2 for the comparative study. (f_{ca}) is the axial stress generated due to the axial compression force on the section.

The moment capacity of the uncracked section, M_c and curvature for the uncracked section, ϕ_c (point 2 on $M-\phi$ curve) is calculated using the stress-strain relationship shown in (Fig. 3).

The moment capacities M_u and M_y are determined by solving the following equilibrium equations:

$$P_u = P_{uc} + P_{us} \quad (9)$$

$$P_y = P_{yc} + P_{ys} \quad (10)$$

$$M_u = M_{uc} + M_{us} \quad (11)$$

$$M_y = M_{yc} + M_{ys} \quad (12)$$

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. Similarly P_{yc} and P_{ys} are the resultant yield forces and M_{yc} and M_{ys} are yield moment capacity of the tubular section.

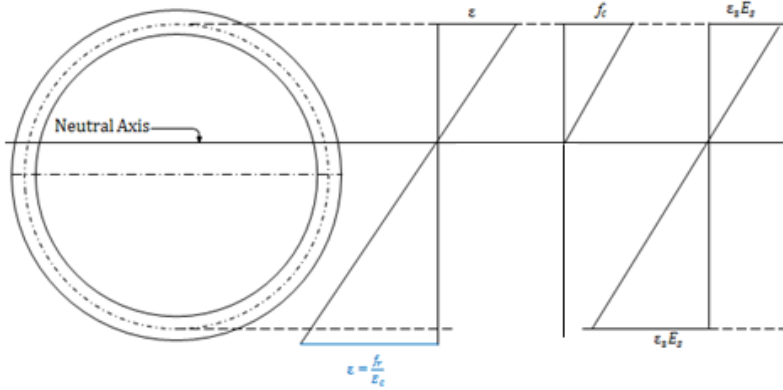


Fig. 3 Stress-strain relationship for concrete and steel for uncracked section

f_r = Modulus of rupture of concrete

(As per IS [3], $f_r = 0.7\sqrt{f_{ck}}$ MPa, as per CICIND [13] and ACI [19], $f_r = 7.5\sqrt{f'_{ck}}$ psi)

E_c = Modulus of elasticity of concrete

(As per IS [3], $E_c = 5000\sqrt{f_{ck}}$ MPa, as per CICIND [13] and ACI [19], $E_c = 57000\sqrt{f'_{ck}}$ psi)

General procedure followed to obtain the points 3 and 4 on the moment-curvature curves are 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 plane section remains plane before and after bending, the value of strain at the geometrical center of each strip ϵ_c and the location of reinforcement ϵ_s is calculated separately. It is to be noted here that to obtain the yield properties of the section, the strain limit at which steel yields is fixed at the level of reinforcement located at extreme tension face and to obtain the ultimate properties of the section, the strain limit at which concrete crushes is fixed at the center of the shell thickness in compression zone. The different design standards gives different strain limits for both conditions.

Step 3: After this, concrete stress at the geometrical center of each strip and the steel stress at the location of reinforcement is calculated individually. It is to be noted here that the stresses for yield and ultimate conditions for these design standards are calculated as per their design recommendations shown in the form of stress block in (Fig. 1 and 2). The concrete stresses f_c are calculated as per equations 13, 14 and 15 for IS: 4998 – 2015, CICIND 2011 and ACI: 307 – 08 respectively for yield and ultimate conditions.

$$f_c = \frac{0.67C_{sf} f_{ck}}{1.5} \left\{ 2 \left(\frac{\varepsilon_c}{0.002} \right) - \left(\frac{\varepsilon_c}{0.002} \right)^2 \right\} \quad (13)$$

Where the short term loading factor can be calculated using (Eq. 1)

$$\begin{aligned} \text{For } \varepsilon_c \leq 0, & \quad f_c = 0 \\ -0.002 \leq \varepsilon_c < 0, & \quad f_c = 1000 \cdot \varepsilon_c \cdot (1 + 250 \varepsilon_c) \cdot (0.85 f_{ck} / 1.5) \\ -0.003 \leq \varepsilon_c < -0.002, & \quad f_c = - (0.85 f_{ck} / 1.5) \end{aligned} \quad (14)$$

$$f_c = 0.85 f_{ck} \times Q \quad (15)$$

Where Q can be calculated using (Eqs. 3 – 8)

Step 4: The concrete stresses at different levels are then multiplied with the 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 and P_y) 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 P_u from the above steps could be matched with the P_u which is an input in the form of axial stress ratio (f_{ca}/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 and M_y).

5. Results and Discussion

The outcomes of the comparative study are discussed here in terms of ultimate strength, ductility and energy absorption in the forms of Tables 2, 3 and 4 and (Fig. 4).

The plotted moment-curvature curves shows that CICIND gives higher values of curvature in all the cases and IS gives the highest values of ultimate moments for almost all cases excepts at higher stress ratios and higher percentage of steel i.e. for stress ratios 0.15 and 0.2 and for percentage of steel 1.5% and 2%. The differences in the following moment-curvature curves are mainly due to the different stress-strain relationships adopted by the three design codes. These differences are further elaborated through the comparison of ultimate moment, ductility ratio and energy absorption obtained from the plotted ($M-\phi$) curves (Fig. 4 (a-e)).

5.1 Comparison of Ultimate Strength

The ultimate moment carrying capacity defines the ultimate strength of RC chimney section when the section is subjected to an axial compressive load associated with a specific neutral axis depth within the section.

It can be seen from (Table 2) that for CICIND and ACI, ultimate capacity of section is increased with an increase in axial stress ratio and percentage of steel but for IS when stress ratio is more than 0.15 and percentage of steel is more than 1.5% the ultimate moment capacity decreases. Although, the differences are not significant between the values obtained from the three design standards. When the stress ratio is small and the percentage of steel is low, the differences between the three design standards are small.

The difference increases with increase in axial stress ratio and percentage of steel. When the RC chimney section is comprised of low axial stress ratio and lightly reinforced with steel bars, the governing condition is the extreme tension condition in which there is hardly any influence from the stress-strain conditions of concrete. The section is subjected to more compression when the axial stress ratio increases and hence, the difference between the three design approaches gets emphasized.

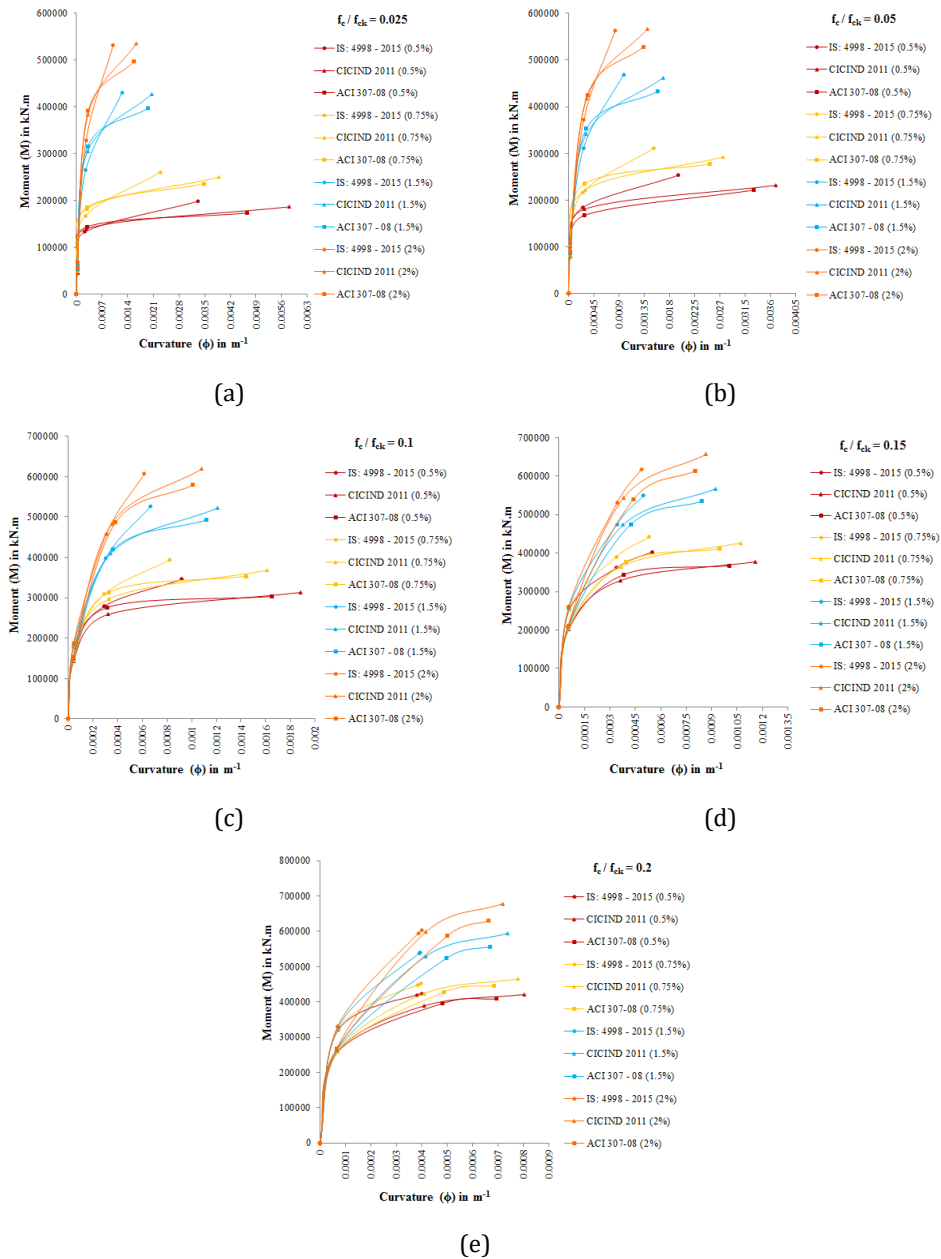


Fig. 4 Moment-curvature ($M-\phi$) interaction curves for stress ratios
 (a) 0.025 (b) 0.05 (c) 0.1 (d) 0.15 (e) 0.2

Table 2. Ultimate strength in kN.m (x 10⁴)

f_{ca}/f_{ck}	p_t %											
	IS: 4998-2015				CICIND 2011				ACI: 307 – 08			
	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0
0.025	19.8	26.0	43.0	53.2	18.6	25.0	42.7	53.6	17.3	23.6	39.7	49.7
0.05	25.3	31.1	46.9	56.4	23.2	29.3	46.2	56.7	22.2	27.7	43.3	52.8
0.1	34.6	39.4	52.7	60.7	31.3	36.8	52.2	61.9	30.3	35.3	49.2	57.9
0.15	40.3	44.1	55.0	61.9	37.7	42.6	56.7	65.7	36.7	41.1	53.5	61.4
0.2	41.9	44.9	54.1	60.3	42.1	46.5	59.4	67.9	40.9	44.6	55.7	62.9

Compared to the other two design standards, the tables also suggest that IS: 4998 – 2015 which gives the highest values of ultimate moments up to lower axial stress ratios is reducing (non-linearly) substantially and drops below the values calculated from ACI: 307 – 08 and CICIND 2011; and at a higher percentage of steel i.e. at 1.5% and 2.0% this decline is substantial.

Contribution of ultimate moment capacity (M_u) of both the materials i.e. steel (M_{us}) and concrete (M_{uc}) in ultimate moment capacity (M_u) of the RC chimney section is also studied separately to understand the above mentioned differences.

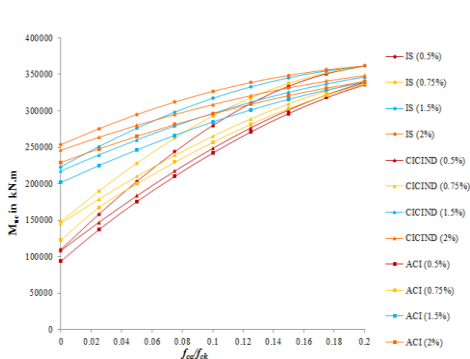


Fig. 5 Comparisons of the contribution of concrete in ultimate moments

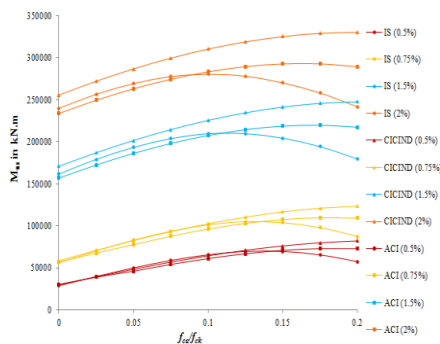


Fig. 6 Comparisons of the contribution of steel in ultimate moments

It is seen from this separate study (Fig. 5) that the contribution of ultimate moment capacity of concrete (M_{uc}) increases with an increase in axial stress ratio (n) for all three methods. It can also be witnessed here that the contribution of concrete obtained using recommendations IS: 4998 - 2015 is higher than the concrete contribution calculated using the recommendations of ACI: 307 - 08 and recommendations of CICIND 2011. This increase in the contribution of concrete calculated using the stress-strain relationship given in IS: 4998 – 2015 is more significant at a lower percentage of steel and higher axial stress ratio.

It can also be noticed here from (Fig. 6) that the contribution of steel in ultimate moment capacity (M_{us}) is reduced at a higher axial stress ratio, when calculated using the recommendations given in IS: 4998 - 2015 as compared to the same calculated using methodologies given in ACI: 307 - 08 and CICIND 2011 and hence, it can be concluded from this separate study that the decrement in the total ultimate moment capacity (M_u) of RC chimney section obtained using the recommendations of IS: 4998 – 2015 in comparison with the other two design recommendations is due to the reduction in the contribution of steel (M_{us}).

It can be concluded that at a lower percentage of steel and up to axial stress ratio of 0.15, the values of M_u is higher for IS: 4998 - 2015 compared to ACI: 307 - 08 and CICIND 2011 as the contribution of concrete (M_{uc}) is higher in this range.

Beyond the axial stress ratio of 0.15, the decrease in ultimate moment capacity calculated from IS: 4998 - 2015 is predominantly because of the decrease in the M_{us} which is significant at a high percentage of steel, as the share of concrete is still increasing but this share is having only a negligible increase in comparison with ACI: 307 - 08 and CICIND 2011 at a higher percentage of steel.

5.2 Comparison of Ductility

The ratio of the deformation at ultimate load to the deformation at yielding is known as the "Ductility Factor". It is an assessment of the RC member's ability to undergo large deformation before failure of the RC member takes place. It can be seen from Table 3 that the RC chimney sections designed using codal provision given in the CICIND 2011 gives higher ductility in all the cases. The values of ductility ratio for the particular percentage of steel decrease with an increase in the axial stress ratios.

Table 3. Ductility ratio

f_{ca}/f_{ck}	p_t %											
	IS: 4998-2015				CICIND 2011				ACI: 307 - 08			
	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0
0.025	14.4	9.67	4.95	3.86	21.5	14.0	6.83	5.24	16.7	12.1	6.48	4.92
0.05	7.81	5.96	3.67	3.04	12.8	9.33	5.36	4.33	11.5	8.66	4.97	3.96
0.1	3.17	2.81	2.21	2.01	5.80	4.88	3.51	3.07	5.17	4.35	3.07	2.64
0.15	1.64	1.57	1.46	1.42	3.20	2.92	2.44	2.27	2.63	2.40	1.99	1.83
0.2	0.95	0.97	1.02	1.03	1.96	1.90	1.78	1.73	1.44	1.41	1.35	1.32

It can be seen from Table 3 that as per Indian Standard design methodologies, the ductility ratio decreases with an increase in the percentage of steel up to stress ratio, $f_{ca}/f_{ck} = 0.15$ but at a higher stress ratio i.e. $f_{ca}/f_{ck} = 0.2$ a little increase in the ductility ratio is observed with increase in the percentage of steel. However, it can be observed from Table 3 that as per design methodologies of CICIND 2011 and ACI: 307 - 08 the ductility ratio decreases with an increase in the percentage of steel for all stress ratios. A remarkable decrease is found in the ductility ratio at stress ratio $f_{ca}/f_{ck} = 0.025$ for all percentages of steel when compared the value of the same with a lower percentage of steel, this decrement is around 32% for all the three design standards.

In comparison with IS: 4998 - 2015, CICIND 2011 gives the higher ductility about 33.0% to 51.5% for $p_t = 0.5\%$, 30.7% to 48.7% for $p_t = 0.75\%$, 27.5% to 42.8% for $p_t = 1.5\%$ and 26.4% to 40.0% for $p_t = 2.0\%$.

In comparison with ACI: 307 - 08, CICIND 2011 gives the higher ductility about 10.4% to 26.5% for percentage of steel $p_t = 0.5\%$, 7.17% to 25.8% for $p_t = 0.75\%$, 5.02% to 24.1% for $p_t = 1.5\%$ and 6.19% to 23.3% for $p_t = 2.0\%$.

5.3 Comparison of Energy Absorption

The energy absorption is the area under the moment-curvature curve up to the chosen curvature level. Here for the comparison, the ultimate level is chosen for all three design standards. It can be seen from the tables below that the RC chimney sections designed using codal provision of CICIND 2011 in comparison with the other two codal provisions i.e. IS: 4998 - 2015 and ACI: 307 - 08 gives higher energy absorption values for all stress ratios and all percentage of steel. It can be seen that the values of energy absorption for the particular percentage of steel decrease with an increase in the axial stress ratios.

If the RC chimney section is designed as per the recommendations of IS 4998 – 2015, the energy absorption values decrease with an increase in the percentage of steel up to stress ratio 0.1 but at a higher stress ratio = 0.15 and 0.2 increase in the values of energy absorption is observed with increase in the percentage of steel as shown in Table 4.

However, it can also be observed from Table 4 that as per design methodologies of CICIND 2011 and ACI: 307 – 08 the values of energy absorption decrease with an increase in the percentage of steel for the stress ratios 0.15 and 0.2. For stress ratio of 0.1, it can be seen that with an increase in the percentage of steel, the value of energy absorption decreases according to CICIND 2011 and increases according to ACI: 307 – 08.

Table 4. Energy absorption in kN

f_{ca}/f_{ck}	p_t %											
	IS: 4998-2015				CICIND 2011				ACI: 307 – 08			
	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0	0.5	0.75	1.5	2.0
0.025	542	476	397	378	952	842	725	704	739	730	671	650
0.05	407	382	340	332	760	703	640	635	647	639	586	579
0.1	263	257	255	250	519	508	509	522	450	452	452	462
0.15	180	183	198	208	378	384	412	435	326	331	354	371
0.2	132	140	160	173	287	299	338	365	240	249	282	304

In comparison with IS: 4998 – 2015, CICIND 2011 gives the higher energy absorption value about 43.0% to 53.9% for percentage of steel (p_t) = 0.5%, 43.4% to 53.2% for p_t = 0.75%, 45.2% to 52.5% for p_t = 1.5% and 46.2% to 52.6% for p_t = 2.0%.

In comparison with ACI: 307 – 08, CICIND 2011 gives the higher energy absorption value about 11.4% to 22.3% for percentage of steel (p_t) = 0.5%, 9.06% to 16.8% for p_t = 0.75%, 7.43% to 16.7% for p_t = 1.5% and 7.62% to 16.9% for p_t = 2.0%.

6. Conclusion

Moment-curvature ($M-\phi$) curves are plotted for hollow thin-walled tubular reinforced concrete sections like RC chimney and TV towers and from these curves, the comparison is carried out between the design recommendations for stress-strain relationship of concrete and steel plotted for IS: 4998 – 2015, ACI: 307 – 08 and CICIND 2011 in terms of strength, ductility and energy absorption.

Considerable disparities exist between the above three design codes which are used to calculate the ultimate and yield moment capacities of the tubular reinforced concrete section, subjected to lateral force and axial compression. These disparities are predominant at higher stress ratio and high steel ratio due to disparities between stress-strain models of concrete and steel adopted by the three codes.

It was concluded that up to the stress ratio of 0.15, M_u calculated from Indian standard is higher compared to the other standards, but the reduction of M_u occurs beyond this stress ratio. Even though the share of concrete is still increasing, the reduction in contribution of steel in M_u induces only a marginal increase in comparison with ACI and CICIND.

The strain limit for concrete in compression specified in the ACI: 307 – 08 and CICIND 2011 (0.003) methods are larger than those specified in the IS: 4998 - 2015 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: 307 – 08 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: 307 – 08 and CICIND 2011 predicting larger strengths than IS: 4998 – 2015, as in this code none of the steel in the tension zone reaches the yield strength whereas, in the ACI: 307 – 08 and CICIND 2011, most of the steel in the tension zone acquires the full yield strength.

It is very evident from the study that CICIND 2011 gives a comparatively higher value of area under the load-deflection of stress strain curve (i.e. moment-curvature curve) as compared to the values obtained as per codal provisions of IS: 4998 – 2015 and ACI: 307-08. As a result, the chimney sections designed using CICIND 2011 exhibit a much higher absorption capacity comparatively. Further, the value of deviation between the deformations (i.e. strain values) at ultimate strength and that at yield strength, when determined by CICIND 2011 is found to be relatively higher than the other two codes and as an outcome of this only, chimney sections designed using CICIND 2011 exhibit higher ductility as well.

Further, the plotted curves in this study ignore the tensile strength of concrete even in calculation of section properties for uncracked section. This results in the curves indicating lower initial slope (initial stiffness) as well as energy absorption. Hereby, the future scope of this study involves the comparison and differences in the stiffness and energy absorption when the tensile strength of concrete is considered and disregarded for the uncracked section. Also, the variations in the three design code further signify the need to compare the results of these theories with the results of experimental study in order to find the methodology which uses most precision in depicting the stress-strain relationship of these materials and hence, to understand the actual scenario that may occur in practice.

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