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Flexural ductility of RC beam sections at high strain rates

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Abstract. Computation of flexural ductility of reinforced concrete beam sections has been proposed by taking into account strain rate sensitive constitutive behavior of concrete and steel, confinement of core concrete and degradation of cover concrete during load reversal under earthquake loading. The estimate of flexural ductility of reinforced concrete rectangular sections has been made for a wide range of tension and compression steel ratios for confined and unconfined concrete at a strain rate varying from 3.3×10^{-5} to 1.0/sec encountered during normal and earthquake loading. The parametric studies indicated that flexural ductility factor decreases at increasing strain rates. Percentage decrease is more for a richer mix concrete with the similar reinforcement. The confinement effect has marked influence on flexural ductility and increase in ductility is more than twice for confined concrete (0.6 percent volumetric ratio of transverse steel) compared to unconfined concrete. The provisions in various codes for achieving ductility in moment resisting frames have been discussed.

Keywords: strain rate; seismic behavior; constitutive relationships; reinforced concrete frames; confinement; ductile detailing

1. Introduction

The Ductility is required in reinforced concrete sections for sufficient deformability to avoid premature failure. This is achieved in various building codes by limiting the tensile reinforcement and ductile detailing of critical sections where plastic hinges are formed. Reinforced concrete is relatively less ductile in compression and shear, the dissipation of energy is best achieved by flexural yielding. The design philosophy of moment resisting frames for dynamic loading is based on the formation of plastic hinges at critical sections of the frames under the effect of load reversals in the inelastic range. The ductile behavior of these hinge sections is ensured if sufficient ductility is available for the RC section. There is international trend for development of performance based seismic design where quantification of ductility is required. For moment resisting frames flexural ductility also named as curvature ductility is usually defined by as the ratio of ultimate and yield curvature and in the past several attempts has been made to quantify curvature ductility.

Park and Paulay (1975) have calculated available flexural ductility factors for doubly reinforced concrete sections assuming linear elastic behavior of compressed concrete up to the

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stage of first yield of the tension reinforcement which he pointed out to result in underestimate of the curvature at first yield. Park and Ruitong (1988) have improved the earlier approach and calculated the flexural ductility factors for reinforced concrete beams with rectangular sections taking into account the possible nonlinear behavior of the unconfined compressed concrete. Comparisons were made for available flexural ductility factors of reinforced concrete sections containing the longitudinal steel ratios permitted by ACI and New Zealand Codes. Al-Haddad (1995) has made a study for the available flexural ductility factors for reinforced concrete sections by taking into account the strain hardening portion of stress strain curve of steel and accounted the effect of strain rate under dynamic loading on steel properties. Lee and Pan (2003) proposed an algorithm and simplified formulas for estimating the relationship between tension reinforcement and ductility of reinforced concrete sections on the similar lines as Park and Ruitong (1988). The effect of concrete confinement has been considered by adjusting the slope of descending branch of stress strain curve of concrete. Kwan *et al*. (2004) studied the effect of concrete strength and confinement on flexural ductility of reinforced concrete beams by analyzing the complete moment curvature behaviour of beam sections cast of different grades of concrete and provided with different amounts of confining reinforcement. Au *et al*. (2009) analyzed the flexural behaviour of prestressed concrete beams with unbounded tendons and parametric studies have been carried out to investigate the effect of various parameters on curvature ductility. Au *et al*. (2011) numerically analyzed the flexural response of reinforced and pre-stressed concrete sections taking into account the non-linearity and stress-path dependence of constitutive materials. It is concluded that the concept of flexural ductility works well for reinforced concrete sections, it can be misleading for pre-stressed concrete sections.

At higher strain rate of loading the mechanical properties of concrete and steel are modified and it is expected that behaviour of structural elements during high strain rate of loading such as earthquake will be affected. Soroushian and Obaseki (1986) made a theoretical study for strain rate effects on the axial-flexural strengths of typical reinforced concrete sections and concluded that the strain rate effect is influenced by the position of the longitudinal steel and confinement provided at the section. The effect of strain rate on seismic response of reinforced concrete frames has been studied by Asprone *et al*. (2012) and concluded that seismic strain rate results in an increase in the structural performance, where only ductile failure mechanisms are considered, but where brittle failure mechanisms are included a decrease of the structural performance is experienced in columns, whereas a slight increase occurs in beams.

An iterative approach has been proposed to include the effect of strain rate on flexural ductility factors of doubly reinforced RC sections by modeling the strain rate effect for concrete and steel which was earlier (Al-haddad 1995) considered for steel only. The proposed algorithm also include confinement effect of core concrete (increase in core stress, increase in core concrete strain and slope of the descending branch), degradation of cover concrete, strain hardening portion of the stress-strain curves of steel, some these factors were considered in earlier studies in parts. The ductility factors are recalculated and parametric studies are made to summarize implications of all the above factors.

2. Comparison of different codes for limiting tensile reinforcement

It is always desirable that ample warning is provided during failure of a reinforced concrete section and this is achieved in Indian Standards and British Standards by restricting the neutral

Table 1 Maximum permissible reinforcement ratio for various grades of concretes and reinforcement for static conditions

f_y (MPa)	415			500		
f_{cs} (MPa)	20	25	35	20	25	35
$\rho_{\max}$ as per IS-456 -2000	0.00957	0.01196	0.01670	0.00760	0.00950	0.0133
$\rho_{\max}$ as per ACI-99	0.01537	0.01927	0.02542	0.01275	0.01600	0.0210

axis depth ratio and thus limiting the maximum area of tension reinforcement. The ACI building code (ACI318-M99) limits on ratio of tension reinforcement (equation) to be less than or equal to 75% of the ratio required for balanced strain condition. The latest version of ACI 318-08 (clause modified in 2002) adopts a new approach according to which the net tensile strain of the extreme tensile steel at nominal strength to be not less than 0.004, the effect of this limitation is to restrict tension reinforcement ratio to be about the same ratio as in the edition of the code prior to 2002 (ACI-318-08).

The maximum tension steel ratio for different concrete grades (M-20, M-25, M-35) and two steel grades (Fe-415 and Fe-500) as per IS 456-2000 and ACI 318-99 is shown in Table 1, it is found that limiting tensile reinforcement ratio is lesser in Indian Standards compared to American Standards in static loading conditions which implies that Indian Standard provide more ductility compared to American Standards in static loading condition.

For seismic design of flexural members, provisions of ACI-318-08 and IS-13920-1993 are similar. In flexural members of continuous structures where seismic design forces are calculated on the basis of energy dissipation in the nonlinear range of response ρ shall not exceed 0.025 and the positive moment strength at the joint face shall not be less than one half of the negative moment strength provided at the face of the joint.

$$\rho \leq 0.025 \quad \rho' \leq 0.0125$$

The New Zealand Code (NZS 3101: 2006) provide additional requirement for tension reinforcement ratio to be of the structural member should not be greater than $\dfrac{0.9 f_{cs} + 10}{6 f_y}$ or $\dfrac{0.9 f_{cs} + 10}{4 f_y}$ based on the research work of Park and Dai 1988.

3. Constitutive behaviour of concrete at high strain rates

The studies on effect of rate of loading /straining on the fundamental properties of concrete have shown that an increase in the rate of loading is accompanied by increase of strength of concrete. The experimental investigations (Hatano *et al*. 1960, Hughes *et al*. 1972, Scott *et al*. 1982, Dilger *et al*. 1984, Ross *et al*. 1995, Grote *et al*. 2001, Yan *et al*. 2007, Xiao *et al*. 2008, Zhang *et al*. 2009) on compressive behavior have shown that concrete compressive strength increases with the rate of loading, however, there is wide scattering in the experimental findings on the magnitude of strength increase with the rate of loading. A study on combined effect of strain rate and low temperature has been made by Filiatrault *et al*. (2001) has indicated significant increase in compressive strength and Young's modulus of concrete when strain rate is increased to 0.1/sec.

Studies on the effects of strain rate on the behaviour of concrete have been directed to develop constitutive models (Scott *et al*. 1982, Dilger *et al*. 1984, Soroushian *et al*. 1986, Meander *et al*. 1989) to represent the complete stress strain curves for plain and reinforced concrete under compression. Scott *et al*. (1982) proposed a constitutive relationship under high strain rate of loading based on experimental investigations at three strain rates (0.33×10^{-5}, 0.167×10^{-2}, 0.0167), The stress-stain relationship was developed by applying a multiplication factor of 1.25 on the peak stress, strain at peak stress and slope of the descending branch of the stress strain curve. Dilger *et al*. (1984) carried out experimental investigations under high rate of loading and presented a constitutive relationship which includes a decrease in strain at peak stress based on his limited experimental investigations. The constitutive models proposed by Soroushian *et al*. (1986a) and Meander *et al*. (1989) were developed by refining the existing empirical expressions for stress strain curves of concrete to include strain rate effects. The strain at failure has not been specified in any of the formulations. The stress strain models proposed by Soroushian *et al*. (1986a) and Meander *et al*. (1989) have the advantage of determining the behaviour of concrete at any specified strain rate of loading.

3.1 Proposed strain rate dependent constitutive model for concrete

The constitutive model used in the present study is similar to the model developed by Soroushian *et al*. (1986) for plain concrete which takes into account the effect of strain rate on compressive strength by a factor K_1 and effect of strain rate on strain at peak stress by a factor K_2. In the present study these factors are defined by CEB 1988 recommendation, which are based on more holistic approach. The stress-strain curve for plain concrete shown in Fig. 1 is a second degree parabola suggested by Hognestad *et al.* (1955) and widely adopted earlier has been used here with two modification factors K_1 and K_2 to take care of strain rate effects. The factor K_3 takes into account the effect of confinement on peak stress, Z defines the slope of falling branch of the stress-strain curve of confined concrete and $\varepsilon_{\max}$ defines the ultimate strain of confined concrete. The constitutive model is defined by the following equations.

$$f_c = f_{cs} K_1 K_3 \left[\frac{2\varepsilon_c}{0.002K_2}\right] - \left(\frac{\varepsilon_c}{0.002K_2}\right)^2 \quad \varepsilon_c \le 0.002K_2 \tag{1(a)}$$

$$f_c = f_{cs} K_1 K_3 \{[1\text{-}Z(\varepsilon_c - 0.002K_2)] \quad \varepsilon_c \le 0.002K_2 \tag{1(b)}$$

Here, K_1 is the dynamic increase factor for compressive strength and K_2 is the ratio of dynamic and static strain at peak stress and defined as follows

$$K_1 = \frac{f_{cd}}{f_{cs}} = \left(\frac{\dot{\varepsilon}_d}{\dot{\varepsilon}_s}\right)^{1.026a} \tag{2a}$$

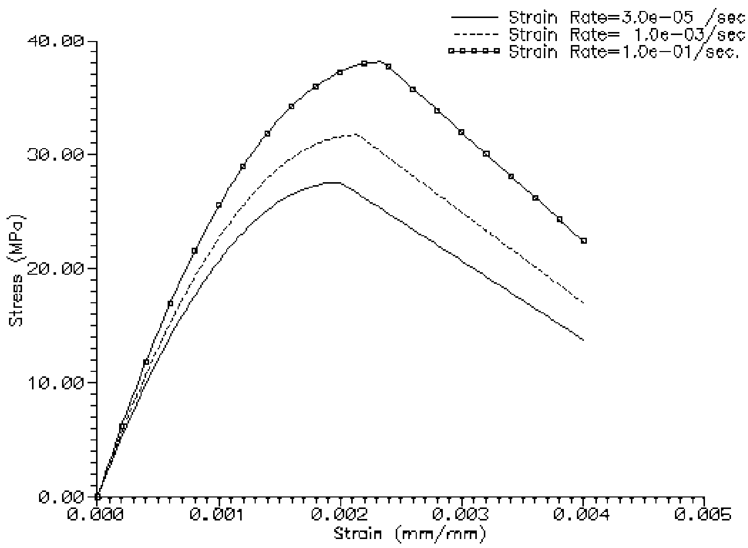


Fig. 1 Stress strain curve of concrete at different strain rates

$$K_2 = \frac{\varepsilon_{od}}{\varepsilon_{os}} = \left(\frac{\dot{\varepsilon}_d}{\dot{\varepsilon}_s}\right)^{0.020} \tag{2b}$$

$$K_3 = 1 + \frac{\rho_s f_y}{f_{cs}} \tag{2c}$$

$$Z = \frac{0.5}{\frac{3+0.29 f_{cs}}{145 f_{cs} - 1000} + \frac{3}{4}\rho_s\sqrt{\frac{h}{s}} - 0.002} \tag{2d}$$

The effect of confinement on ultimate strain is given by the following equation (Scott *et al.* 1982)

$$\varepsilon_{\max} = 0.004 + 0.9\rho_s\left(\frac{f_y}{300}\right) \tag{2e}$$

Here, $\alpha = (5+3f_{cs}/4)^{-1}$ $\quad and\ \dot{\varepsilon}_s = 3.0x10^{-5}/\sec.$

4. Constitutive behaviour of reinforcing steel at high strain rates

Stress-strain curve of steel used as reinforcement in concrete depends upon the loading/ strain rate. The yield stress and ultimate strength of steel will increase (Cowell 1965, Mahin *et al.* 1983, Staffer *et al.* 1985, Asprone *et al.* 2009, Cadoni *et al.* 2011) and strain corresponding to these stresses will either increase or remain constant with increasing strain rate. However the modulus of elasticity (Cowell 1965, Mahin *et al.* 1983, Cadoni *et al.* 2011) will not be significantly influenced by the rate of straining. Uniaxial dynamic experiments (Cowell 1965, Mahin *et al.* 1983, Staffer *et al.* 1985) conducted on steel bars have shown that steel with lower yield strength are more sensitive to strain rate variations compared to steel with higher yield strength. Filiatrault *et al.* (2001) studied the effect of strain rate and low temperature and found that there is moderate increase in yield strength and ultimate strength of reinforcing steel. Soroushian and Ki-bong choi (1987) reviewed experimental results on properties of reinforcing bars. The study by Soroushian Ki-bong choi (1987) indicated that there is wide scatter in the reported test results and following expression for dynamic yield (f_{yd}) and ultimate strength (f_{ud}) of steel as function of strain rate ($\dot{\varepsilon}$) and static yield strength (f_y) have been proposed by encompassing the available experimental results.

$$f_{yd} = 3.1 + 1.2 f_y + (0.65 + 0.05 f_y)\log\dot{\varepsilon} \tag{3a}$$

$$f_{ud} = -20.0 + 2.5 f_y + (-2.4 + 0.12 f_y)\log\dot{\varepsilon} \tag{3b}$$

The constitutive model for steel used in the study considers reinforcing steel from linear elastic to strain hardening material and strain rate effects are incorporated by Eqs. 3a and 3b. The model was used in the earlier study by Al-Hadad *et al.* 1995 for evaluation curvature ductility. The three strain ranges are expressed as follows.

$$f_s = E_s \varepsilon_s \quad \text{for} \quad 0 \le \varepsilon_s \le \varepsilon_y \tag{4a}$$

$$f_s = f_y \quad \text{for} \quad \varepsilon_y < \varepsilon_s \le \varepsilon_{sh} \tag{4b}$$

$$f_s = f_y + \gamma(f_{su} - f_y) \quad \text{for } \varepsilon_s > \varepsilon_{sh} \tag{4c}$$

$$\gamma = \frac{AX + BX^2}{1 + CX + DX^2} \tag{5a}$$

$$X = \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_{sh} - \varepsilon_{sh}} \tag*{5(b)}$$

The constants A, B, C, D are determined by Wang *et al.* (1978) using regression analysis and reproduced here. For deformed bars of yield strength of 415 MPa (60 ksi), modulus of elasticity of 200000 MPa (29760 ksi) the above parameters are as follows.

$$f_{su} = 721.7(104.6\,ksi),\ \varepsilon_s = 0.002,\ \varepsilon_{sh} = 0.0091,\ \varepsilon_{su} = 0.0729, A = 1.748,\ B = 0.1736,$$
$$C = -0.2517, D = 1.736$$

For strain rate modeling f_y is replaced with f_{yd} and f_{su} with f_{ud}. Stress strain curve obtained by the above equations is shown in Fig. 2 for three different strain rates.

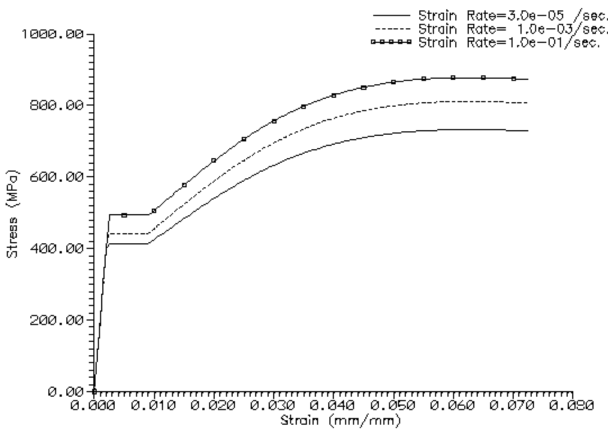


Fig. 2 Stress strain curve of steel at different strain rates

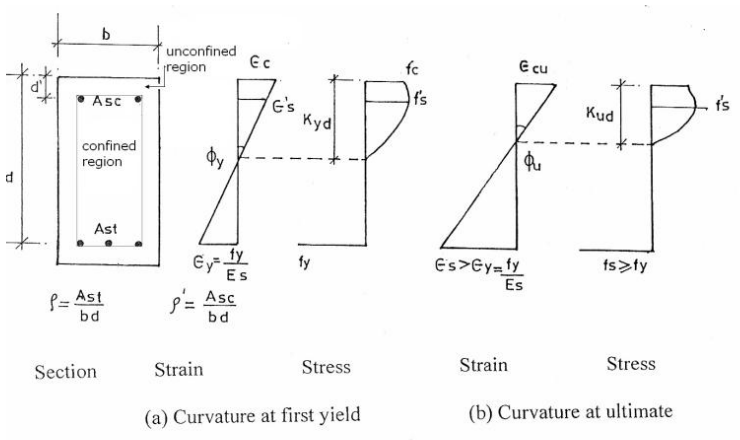


Fig. 3 Doubbly reinforced concrete beam section with flexure

5. Computation of curvature at first yield and ultimate

It is assumed that strain distribution in the beam along the depth is linear and tensile strength of concrete is negligible. The crushing strain of concrete in the analysis for unconfined concrete has been taken equal to 0.004 and for confined concrete it has been calculated as per Eq. 2e. It has been assumed that concrete cover will get degraded completely when ultimate moment capacity is reached. Therefore while calculating the ultimate curvature (ultimate neutral axis depth) internal forces in the concrete in unconfined region has not been accounted. The degradation of concrete cover will begin after first yielding of the tension reinforcement and therefore yield curvature will not be affected. The curvature at first yield is the curvature of the section (Fig. 3(a)) when the tensile reinforcement reaches the yield strain. This can be written as

$$\varphi_y = \frac{f_y / E}{d(1-K_y)} = \frac{\varepsilon_y}{d(1-K_y)} \qquad 6(a)$$

Table 2 Comparison of some results with earlier studies

Design Condition	$\frac{\rho'}{\rho}$	Park and Ruitong 1988	Al-Haddad, 1995		Present Study	
			Static Case	$\dot{\varepsilon}$ =0.05	Static Case	$\dot{\varepsilon}$ =0.05
Ductility factor at ρ = 0.025	0.50	4.0	NA	2.75	3.87	3.40
Maximum permissible ρ to achieve ductility factor=8	0.50	0.0116	0.0080	0.0065	0.0080	0.0066
	0.75	0.0150	0.0100	0.0090	0.090	0.0070

Based on geometry, value of neutral axis depth is obtained as follows

$$k_y = \frac{\varepsilon_{ce}}{f_y / E_s + \varepsilon_{ce}} \tag{6b}$$

The curvature at ultimate is the curvature when concrete strain in the compression region at top fiber reaches to ultimate strain in the concrete. This can be written as

$$\varphi_u = \frac{\varepsilon_{cu}}{K_u d} \tag{7}$$

The flexural ductility factor df is defined as

$$df = \frac{\phi_u}{\phi_y} = \frac{\varepsilon_{cu}(1 - K_y)}{\varepsilon_y K_u} \tag{8}$$

Here in this ε_{cu} (equal to 0.004 for unconfined and $\varepsilon_{\max}$ for confined concrete) and ε_y (yield stress of steel) are the properties of the concrete and steel used in the section. K_u and K_y are determined by using an iterative approach as follows.

(i) The neutral axis depth is assumed (initial value K_y and K_u is taken as 0.08 and 0.04) and strain values in steel and concrete are evaluated as per the linear strain diagram.

(ii) The stresses in concrete and steel and strain-strain parameters corresponding to strain rate and taking the effect of confinement at required locations are determined as per the constitutive relationships Eq. (1, 5). The compression zone is divided into two parts confinement region and unconfined region and stress-strain parameters are separately calculated in these regions.

(iii) Now internal forces in concrete (separately calculated for confined and unconfined zones) and steel in tension and compression zones are evaluated and for equilibrium these should be equal with acceptable degree of accuracy. In case of difference, the neutral axis depth ratio is increased *(by 0.001)* and the procedure from (i) to (iii) is repeated till the force equilibrium is obtained.

6. Parametric studies

Flexural ductility of concrete section for different percentage of tension and compression steel ratios has been studied using strain rate sensitive properties of concrete and steel, confinement of core concrete and degradation of cover concrete during load reversal under earthquake loading.

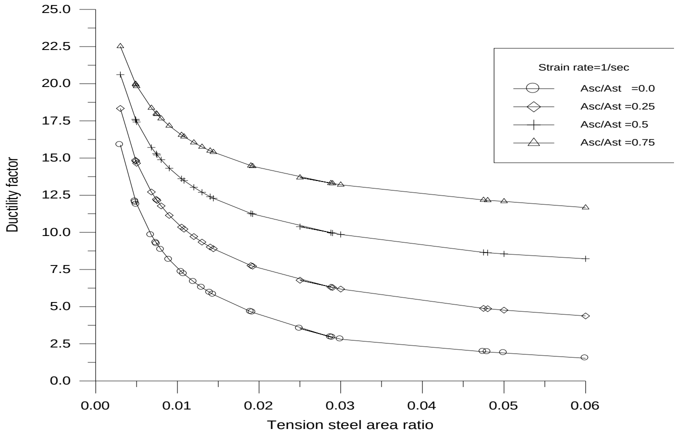
The study has been made for three grades of concrete (M-20, M-25 and M-35) and one grade of steel (Fe-415 TMT bars) widely used in Indian Construction Industry. Comparison of results with earlier studies for the selected cases for unconfined concrete with ultimate strain equal to 0.004 are shown in Table 2. The comparison indicate that the flexural ductility estimated by Park and Ruitong (1988) may have been overestimated because he has not considered the strain hardening in steel and strain rate effects. The results of the other investigator compare well with the results of present study.

6.1 Effect of compression reinforcement on flexural ductility

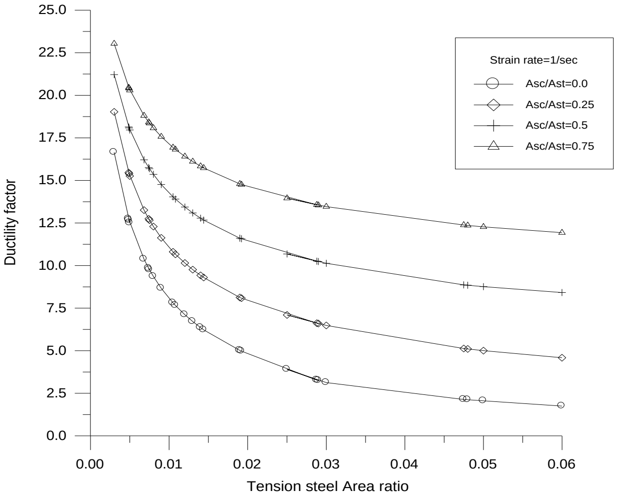
It has been established earlier (Park and Ruitong 1988) that doubly reinforced concrete section has more flexural ductility factor. A parametric study has been made on flexural ductility factors by varying tension reinforcement percentage corresponding to three compression steel ratios ($\rho^{'}/\rho$ = 0.25, 0.50, 0.75) for three grades of concretes (M-20, M-25, M-35) with deformed bars (Fe-415). The study has been made for unconfined concrete by taking into account degradation of concrete cover, corresponding to strain rate of 1/sec. The results are shown in Figs. 4(a, b, c) for three grades of concrete (M-20, M-25, M-35) with deformed bars (Fe-415). It is seen from the figures that increasing the compression reinforcement ratio results in significant increase in curvature ductility up to 2.0% of tension reinforcement. For tension reinforcement lesser than 2.0%, gap between the curves corresponding to different $\rho^{'}/\rho$ ratios reduce. It is found that for a highly reinforced concrete section with tension reinforcement of 2.5%, the flexural ductility factor increases to 193, 174 and 145 percent for M-20, M-25 and M-35 grade of concrete respectively at $\rho^{'}/\rho$ = 0.5 compared to sections without compression reinforcement. At a medium tension reinforcement corresponding to 1.05%, the flexural ductility factor increases to 85, 80 and 70 percent for M-20, M-25 and M-35 grade of concrete respectively at $\rho^{'}/\rho$ = 0.5 compared to sections without compression reinforcement. For a highly under reinforced concrete section with tensile reinforcement equal to 0.3 percent, increasing the compression reinforcement ratio from 0.0 to 0.5, ductility factors increase by 30, 27 and 23 percent for M-20, M-25 and M-35 grade of concrete respectively. Further it is to be mentioned here that highly under-reinforced sections have ductility more than 15 and compression reinforcement may not be required except for shrinkage purpose.

6.2 Effect of strain rate of loading on flexural ductility

As found experimentally at higher strain rate of loading there is increase in yield and ultimate strength of steel increases which may reduce the flexural ductility factors. At higher loading rates, there is increase in concrete compressive strength and strain at peak stress, which may increase the ductility factors, however parametric study indicate that overall effect of concrete and steel properties at higher strain rates is to decrease the flexural ductility factors. The parametric study has been made for the deformed steel bars of yield strength 415 MPa and three grades of concrete for unconfined and confined condition and results shown in Figs. 5(a)-(c) and 6(a)-(c) It is seen from the figures that there is decrease in curvature ductility (gap between curves reduces at lower tension reinforcement) for unconfined and confined concrete with increase in strain rate. It is found that for M-20 grade concrete with deformed bars a reduction of 15.0, 20.3 and 23.4 percent results for tensile reinforcement percent of 0.3, 1 and 2.5 respectively for the strain rate of 1/sec.



(a)



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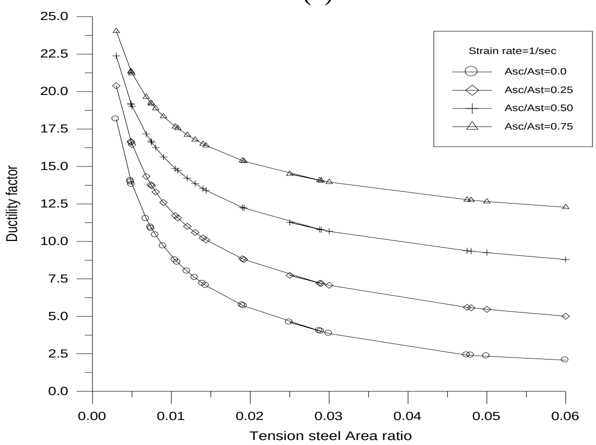
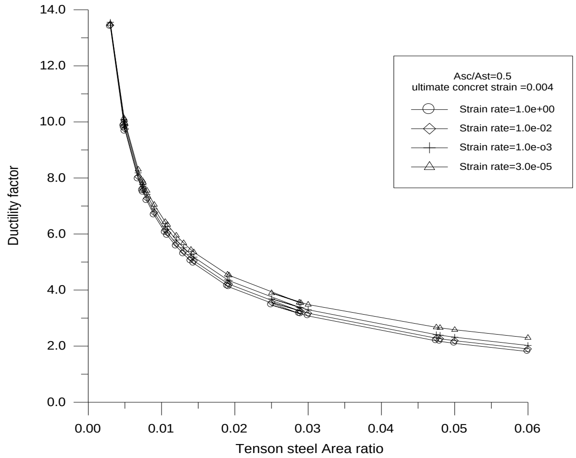
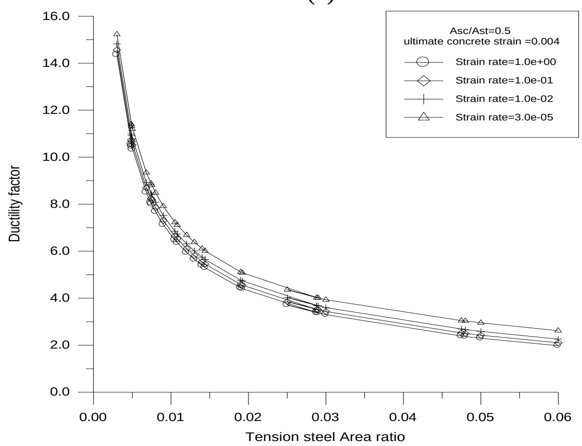


(c)

Fig. 4 Varation of ductility factor for M-35 grade concrete and Fe-415 steel (TMT) for different compression steel considering a strain rate, confinement and degradation of concrete cover



(a)



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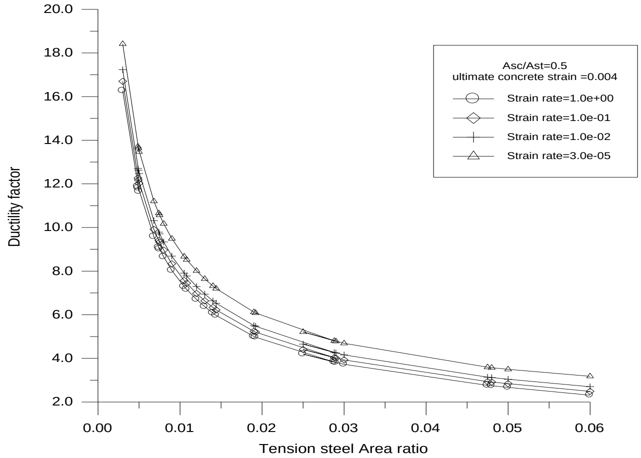
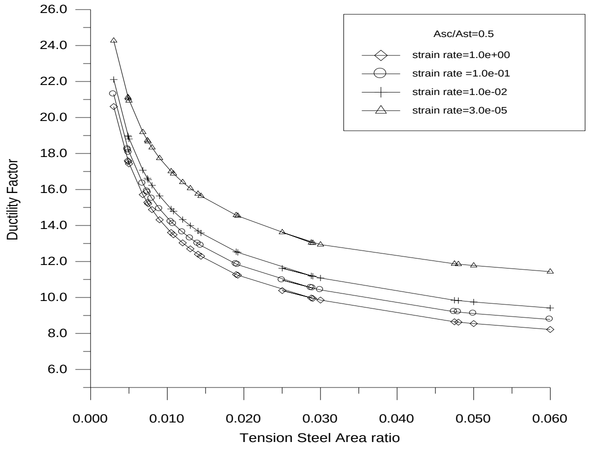
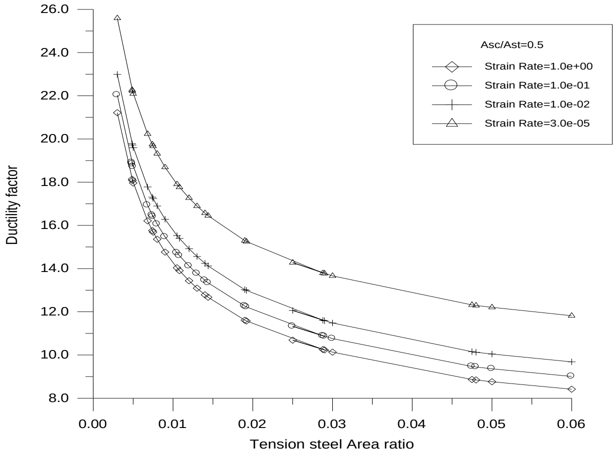


(c)

Fig. 5 Varation of ductility factor for M-35 grade concrete and Fe-415 steel (TMT) for different compression steel considering a strain rate, confinement and degradation of concrete cover



(a)



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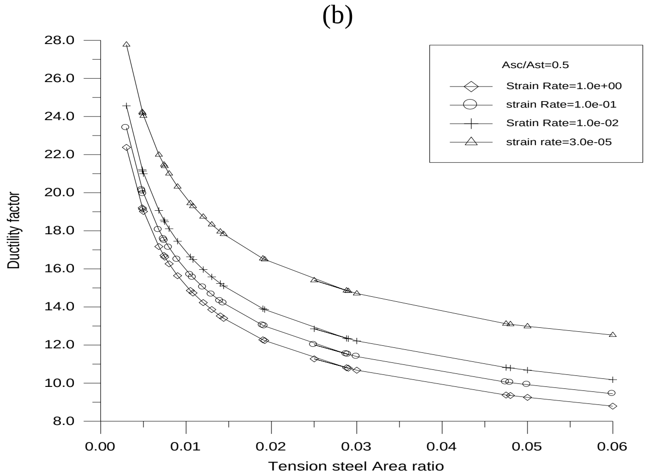

(c)

Fig. 6 Varation of ductility factor for M-35 grade concrete and Fe-415 steel (TMT) for different compression steel considering a strain rate, confinement and degradation of concrete cover

Table 3 Variation of flexural ductility factor for three grades of concrete for ρ_{max} seismic condition

Strain Rate 1/sec	Flexural ductility factor for different grades of concrete		
	20 MPa	25 MPa	35 MPa
Static $\dot{\varepsilon}$ =3.0 × 10^{-5}	13.62	14.28	15.39
$\dot{\varepsilon}$ =1.0 × 10^{-2}	11.62	12.06	12.84
$\dot{\varepsilon}$ = 1.0	10.38	10.68	11.27

compared to static strain rate. The decrease in flexural ductility factors at for M-25 concrete is 17.2, 21.6 and 25.1 for tensile reinforcement percent of 0.3, 1 and 2.5 respectively for the strain rate of 1/sec. compared to static strain rate. Similarly the percent reduction in flexural ductility factor values for M-35 grade concrete is found to be 19.3, 23.6 and 26.8 percent for tensile reinforcement percent of 0.3, 1 and 2.5 respectively for the strain rate of 1/sec. compared to static strain rate. The reduction in flexural ductility factors is more for higher strength concretes for the peak strain rate 1/sec. considered in the analysis. The strain rate model for concrete used in the analysis considers the experimental fact that weaker concretes are more strain rate sensitive and the increase in compressive strength is comparatively more resulting in lower overall decrease in flexural ductility factor values.

6.3 Effect of confinement on flexural ductility

The concrete gets confined within the shear stirrups and this result in the increase of the peak stress, strain at peak stress and ultimate strain. As seen from the parametric studies for M-20 grade unconfined and confined concrete from Figs. 5(a) and 6(a) that there is substantial increase in the flexural ductility factor. Similar trend follows for M-25 grade and M-35 grade concrete. The parametric study has been performed for percentage volumetric transverse steel ratio of 0.4, 0.6, 0.8 and 1.0 percent for three grades of concrete and Fe-415 steel. It is found that for M-20 grade concrete with increase in volumetric transverse steel ratio from 0.4% to 1%, the increases the flexural ductility factor is 131.0 and 146.8 % for RC sections with longitudinal steel ratio of 1.05% and 2.5% respectively. Similarly for M-25 grade concrete with increase in volumetric transverse steel ratio from 0.4% to 1%, the increase in flexural ductility factor by is 128.0 and 143.9 percentage for RC sections with longitudinal steel ratio of 1.05% and 2.5% respectively. Similar variation has been obtained for M-35 grade concrete.

6.4 Flexural ductility under different specific codal provisions

The variation of ductility factor for three concretes for ρ_{max} specified in ACI-318-08 and IS-13920-1993 for seismic condition with confinement effect and by varying the strain rate is shown in Table 3, it is found that ductility factor varies from 13.26 to 15.28 for the three concretes at slowest strain rate and the ductility factor is more for higher strength concretes. At higher strain rate ductility factor decreases with a decrease of 23.8, 25.2 and 26.7 for three grades of concrete (M-20, M-25, M-35) respectively at the highest stain rate of 1/sec.

7. Conclusions

An analytical model for computation of flexural ductility of beam sections has been proposed by taking into account the strain rate dependent properties of concrete and steel, effect of concrete confinement and degradation of concrete cover during load reversal in earthquake loading. The model considers the strain hardening in steel and softening branch in concrete. Parametric studies have been made for the variation of flexural ductility factors as a function of strain rate of loading, percent tensile reinforcement for different compression and tension steel ratio, percentage volumetric transverse reinforcement. Various code provisions for providing ductility in RC sections have been discussed. Following are the conclusions drawn from the study.

- The parametric studies on flexural ductility factor have shown that consideration of strain rate dependent properties of concrete and steel decreases the flexural ductility at higher strain rate of loading which occurs during earthquake loading. For the three concretes (M-20, M-25, M-35) considered in the analysis, it is found that overall effect of strain rate decreases the flexural ductility factor between 15 to 19.3% (0.3% tension reinforcement), between 20.3 to 23.6% (1.05% tension reinforcement) and between 23.4 to 26.8% (2.5% tension reinforcement). For higher strength concretes reduction is more pronounced.
- Compression reinforcement increases the ductility factor significantly for highly reinforced concrete sections. The study indicated that for compression reinforcement ratio (ρ' / ρ) equal to 0.5, the flexural ductility factor increases by 194, 173, 145 % for M-20, M-25 and M-35 grade for tensile reinforcement percent of 2.5% (highest permissible tensile reinforcement in seismic loading condition as per ACI-318-08 and IS-13920-1993).
- The confinement of core concrete by shear stirrups has a marked influence on the flexural ductility and increase in ductility is manifold. The ductile detailing by IS-13920-1993 ensures a minimum volumetric transverse reinforcement percent of approximately 0.6% (obtained in various projects in which author is associated) and this ensures a ductility factor more than 10 for maximum permissible ratio of tensile steel in seismic loading conditions. The flexural ductility factor varies from 10.38 to 11.27 for M-20, M-25, M-35 grade of concrete at highest strain rate of loading (1/sec.) occurring during severe earthquake for maximum permissible reinforcement ratio for seismic loading condition as per IS-13920-1993 and ACI-318-08 (ρ_{max} = 0.025 and $\rho' / \rho \geq 0.5$**).**

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