Ninth Symposium on Earthquake Engineering (9SEE - 90), Roorkee, December 14-16, 1990, Vol. 1

Suggested Reinforcement Requirements for Flexural Members in IS : 4326

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SYNOPSIS

IS:4326-1976 provisions for reinforced concrete flexural members are reviewed. The minimum reinforcement requirement of the code is inadequate and needs revision. Expressions are given for maximum reinforcement that ensure member ductility of 5. Suggestions are given for shear design of flexural members. An approximate method is given to compute plastic moment capacity.

INTRODUCTION

An earthquake resistant structure should be ductile. During a severe earthquake, it should be capable of undergoing inelastic deformations. Such structures are subjected to less earthquake force and they dissipate seismic energy more efficiently. A structure can be ductile only if its constituent members are ductile. This requires certain restrictions to be imposed on the provision of reinforcement in the members. Hence, IS:4326-1976 (6) provides specifications for provision of reinforcement in flexural members. Many of these provisions are either incomplete or are based on assumptions that are not consistent with IS:456-1978 (7). In this paper, proposals on these requirements are put forward.

MATERIAL SPECIFICATIONS

IS:4326-1976 does not specify the minimum grade of concrete to be used for flexural members. Hence, as per IS:456-1978, the minimum grade of concrete shall be M15 (cube strength, f_{ck}, as 15 MPa). As a high grade of concrete imparts greater ductility to the member, ACI and UBC codes (1, 13) specify that concrete having cylinder strength, f_{cp}, of 3000 psi (equivalent f_{ck} = 26 MPa) and above be used in zones of high seismic risk. Taking Indian conditions into consideration, the minimum grade of concrete may be specified as M20 for seismic zones IV and V. A high yield strength of steel significantly reduces the member ductility. Hence, it is recommended that the use of steel reinforcement of grade Fe 500 (yield strength, f_y, as 500 MPa) be not allowed in seismic zones IV and V.

MINIMUM FLEXURAL REINFORCEMENT

IS:4326-1976 requires that at least two bars be provided, at the top and bottom, throughout the length of the member. The steel ratio, ρ, on either face, should be greater than 0.06 f_{ck}/f_y. This requirement is based on the consideration that tension reinforcement must not fracture before the crushing strain of concrete is reached (3). This requires the non-dimensional depth of the neutral axis, Ku, to be

$$Ku \geq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_h} \tag{1}$$

where ε_{cu} is the crushing strain of concrete and ε_h is the strain at which steel begins to strain harden. This gives the minimum reinforcement ratio for a singly reinforced section as

$$\rho_{min} \geq \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_h} \right) \frac{f_{cu}}{f_y} \tag{2}$$

where f_{cu} is the average concrete stress at the ultimate condition. The value of 0.06 f_{ck}/f_y is based on ε_{cu} = 0.003, ε_h = 0.024, f_{cu} = 0.7 f_{cp} and f_{cp} = 0.785 f_{ck}. However, per IS:456-1978, ε_{cu} = 0.0035 and f_{cu} = 0.81 f_{cp}. Further, the value of ε_h varies from 0.015 to 0.030 (3). Taking ε_{cu} = 0.0035, f_{cu} = 0.81 f_{cp} and f_{cp} = 0.80 f_{ck} (8), the minimum reinforcement ratio, ρ_{min}, works out as 0.12 f_{ck}/f_y for ε_h = 0.015; and as 0.07 f_{ck}/f_y for ε_h = 0.030.

Another criterion is also used to decide the minimum reinforcement. The flexural strength of the member with minimum tension reinforcement must be greater than the bending moment required to crack the member, considering it as a plain concrete beam (2, 12, 14). Taking the modulus of rupture, f_{cr}, as 0.7 $\sqrt{f_{ck}}$ and the ratio of effective to overall depth as 0.90, the minimum reinforcement ratio for rectangular beams varies from 0.67/f_y to 1.08/f_y for concrete grade varying from M15 to M40, respectively. IS:456-1978 specifies this ratio as 0.85/f_y. However, most beams in monolithic reinforced concrete structures behave as T beams. For such beams, with flange in compression, the minimum reinforcement ratio varies from $1.0/f_y$ to $1.7/f_y$ for concrete grade varying from M15 to M40, respectively.

The minimum reinforcement ratio, obtained by both the criteria mentioned above, is directly proportional to the ratio f_{ck}/f_y. As per IS:456-1978 the mean strength of concrete, f_m, is given as f_{ck} + 1.65 s. Here s is the standard deviation, which varies from 3.5 for M15 concrete to 6.6 for M40 concrete, for good degree of control (11); and is even higher for fair degree of control. Thus, concrete on site will have strength appreciably greater than f_{ck}. Steel, however, does not show much standard deviation in strength. Therefore, the strength of concrete used in calculating the minimum reinforcement should be taken greater than the characteristic value. Taking all these factors into consideration, the minimum steel provision in IS:456-1978 and IS:4326-1976 needs to be revised.

MAXIMUM FLEXURAL REINFORCEMENT

IS:4326-1976 stipulates that the maximum tensile steel ratio, ρ_{max}, on any face, at any section, must not exceed $\rho_c + 0.19\ f_{ck}/f_y$ for mild steel reinforcement; and $\rho_c + 0.15\ f_{ck}/f_y$ for cold worked deformed bars; where ρ_c is the steel ratio on the compression face. These expressions are meant to ensure member ductility of 5 (10). They are obtained by ensuring that strain in tension steel is $5\varepsilon_y$ when concrete reaches its crushing strain; where ε_y is the yield strain of steel. However, this does not ensure rotational ductility of 5 for the member because the entire expression for rotational ductility, ϕ_u/ϕ_y, is not set equal to 5; where ϕ_u is the deformation at ultimate and ϕ_y is the deformation at yield.

A study has been carried out to estimate the maximum ratio of tension steel that imparts rotational ductility (ϕ_u/ϕ_y) of 5 to the flexural member. On the basis of this study , the authors suggest the following expression for ρ_{max}:

$$
\begin{aligned}
\rho_{max} &\le \rho_c + 0.00071\ f_{ck} && (\text{for } f_y = 250 \text{ MPa}) \\
\rho_{max} &\le 0.75\ \rho_c + 0.00034\ f_{ck} && (\text{for } f_y = 415 \text{ MPa}) \qquad (3) \\
\rho_{max} &\le 0.55\ \rho_c + 0.00024\ f_{ck} && (\text{for } f_y = 500 \text{ MPa})
\end{aligned}
$$

COMPRESSION REINFORCEMENT

ACI code specifies that the positive moment capacity at a joint face must be at least one half of the negative moment capacity at that joint face. This is to cater for moment reversal. This clause may also be specified in IS:4326.

SHEAR REINFORCEMENT

The clause for shear reinforcement for flexural members (cl.7.2.5) in IS:4326-1976 reads: "The web reinforcement in the form of vertical stirrups shall be provided so as to develop the vertical shears resulting from all ultimate vertical loads acting on the beam plus those which can be produced by the plastic moment capacities at the ends of the beam. The spacing of the stirrups shall not exceed d/4 in a length equal to 2d near each end of the beam and d/2 in the remaining length." Note that the clause neither defines the term 'plastic moment capacity' nor gives a method for its calculation.

The maximum shear force developed in a flexural member is directly proportional to the plastic moment capacity at its ends. The maximum probable plastic moment capacity, due to material strength being larger than that specified, must be used for calculation of the design shear force. The tension steel may also strain harden during a severe earthquake. Considering these factors, it is suggested that the plastic moment capacity be calculated using:

(a) The partial safety factor for material strength, γ_m, as 1.0 for steel and 1.3 for concrete (12).

(b) The characteristic stress strain curve of steel be used up to the yield strain, ε_y. Beyond the yield strain, the stress in the tension steel be taken as 1.25 f_y while that in compression steel be taken as f_y only.

Also, the ultimate vertical load for calculating the design shear force be taken as 1.2 times the sum of dead and live loads on the span.

The resulting formulation for plastic moment capacity of doubly reinforced sections involves quadratic equat_ons and is not suitable for design office application. Hence, approximate methods are used to estimate the plastic moment capacity. Refs. (4, 5) neglect the compression steel while calculating the plastic moment capacity of doubly reinforced sections. The resulting plastic moment capacity is then increased by 5 percent to account for compression steel. This method gives reasonable results only for sections having a low percentage of tension steel. Therefore, the following alternative method has been developed to calculate the plastic moment capacity.

Singly Reinforced Section

The plastic moment capacity is given by

$$\frac{M_p}{bd^2} = 1.25\ f_y\ P_t\ (1.0 - 0.416\ Ku) \qquad (4)$$

where P_t = ratio of tension steel (A_{st}/bd); b = breadth of member; d = effective depth of member; and Ku = non-dimensional depth of neutral axis, which is given by

$$Ku = \left[\frac{1.25\ f_y\ P_t}{0.4172\ f_{ck}} \right] \qquad (5)$$

Doubly Reinforced Section

Let P_{tb} be the tension steel ratio (A_{st}/bd) for a singly reinforced balanced section as per provisions of IS:456-1978. Similarly, let P_{cb} be the compression steel ratio (A_{sc}/bd) for a doubly reinforced balanced section as per IS:456-1978 for a tension steel ratio of P_t. If the tension steel ratio of the section, P_t, is less than P_{tb}, the section is under-reinforced, irrespective of the compression steel ratio present. For a section that has tension steel ratio, P_t, greater than P_{tb}, if the compression steel ratio, P_c, is greater than P_{cb}, the section is under-reinforced; otherwise it is over-reinforced. Let the total tension steel ratio, P_t, consist of two parts, P_{t1} and P_{t2}. Here, P_{t2} is assumed to balance the force due to the compression steel only and P_{t1} is equal to the difference of P_t and P_{t2}. Further, let the plastic moment capacity, M_p, be comprised of two parts, M_{p1} (due to concrete and P_{t1}) and M_{p2} (due to P_c and P_{t2}). The following two cases arise depending on whether the section is under-reinforced or over-reinforced.

Case I Under-Reinforced Section

(i) Obtain f_{sc} from Table 1. Stress values in Table 1 have been obtained by multiplying stress values of Table F of SP:16 by 1.15 to make the partial safety factor for material strength of compression steel as 1.0 against 0.87.

(ii) Find $P_{t2} = \left[\dfrac{f_{sc}\ P_c}{1.25\ f_y} \right]$ (6)

If P_{t2} is greater than P_t, set $P_{t2} = P_t$.

(iii) Calculate $P_{t1} = P_t - P_{t2}$

(iv) Obtain M_{p1} from Eq. (4) by substituting P_{t1} as P_t.

(v) Obtain M_{p2} from the following

$$\frac{M_{p2}}{bd^2} = 1.25\ f_y\ P_{t2} \left(1.0 - \frac{d'}{d} \right) \tag{7}$$

(v) $M_p = M_{p1} + M_{p2}$ (8)

Case II Over-Reinforced Section

(i) Obtain M_{p1} from Eq. (4) by substituting P_{tb} as P_t.

(ii) Obtain M_{p2} from the following

$$\frac{M_{p2}}{bd^2} = f_{sc}\ P_c \left(1.0 - \frac{d'}{d} \right) \tag{9}$$

where f_{sc} is taken from Table 1.

(iii) $M_p = M_{p1} + M_{p2}$

A parametric study has indicated that the approximate method underestimates the plastic moment capacity of sections that have tension steel ratio less than P_{tb}, by not more than 5 percent. For doubly reinforced sections that are under-reinforced as per IS:456-1978, the approximate method underestimates the plastic moment capacity by a maximum of 2.5 percent. For doubly reinforced sections that are over-reinforced as per IS:456-1978, the approximate method may underestimate the plastic moment capacity by a maximum of 17 percent. However, such sections are not to be used in aseismic design due to their poor ductility.

SUMMARY AND CONCLUSIONS

The specifications of IS:4326-1976 for flexural members are reviewed. It is proposed that for seismic zones IV and V, the minimum grade of concrete be specified as M20 while use of grade Fe 500 steel be not allowed. The provisions on minimum flexural reinforcement specified by IS:4326-1976 and IS:456-1978 need to be revised. The maximum flexural reinforcement must ensure member rotational ductility of at least 5. Based on this criterion, simple expressions for maximum flexural reinforcement are proposed. Provisions of IS:4326-1976 for shear design are incomplete. It is suggested that the plastic moment capacity be calculated by assuming partial safety factors for material strength as 1.0 and 1.3 for steel and concrete, respectively; and stress in the tension reinforcement as 1.25 f_y on yielding. The ultimate vertical load for calculation of design shear force may be taken as 1.2 times the sum of dead and live loads on the span. An approximate method is developed to calculate plastic moment capacity based on the above parameters.

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TABLE 1: Stress in Compression Reinforcement, f_{sc} (N/mm^2), for a Doubly Reinforced Section*

f_y (N/mm^2)	d'/d			
	0.05	0.10	0.15	0.20
250	250	250	250	250
415	408	406	393	378
500	488	474	454	426

* To be used in the approximate method for calculating plastic moment capacity.