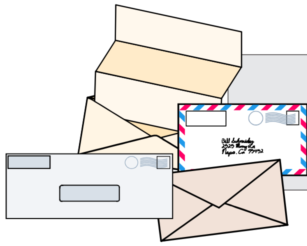

Letters to the Editor

Shortcomings in structural design provisions of IS 456 : 2000

I read with interest the 'Point of View' titled "Shortcomings in structural design provisions of IS 456 : 2000", by Dr C.V.R. Murty", published in the February 2001 issue of *The Indian Concrete Journal*. The paper is critical about several design provisions of the new code IS 456 : 2000, which in my opinion needs discussion. I would like to make the following observations on some of these points.

(i) Firstly, there has been no change in the codal provisions of the new code from the earlier 1978 version, regarding limit-state method of design and several other design clauses. For flexure by limit-state method, clauses 38.1 and G.1 of Annex G of the code are very clear that rectangular sections are to be designed as under-reinforced concrete sections only. For the limiting value of '$x_{u,max}/d$', which is specifically given in the footnote of the clause 38.1, the section will be a balanced design with strains of concrete and steel reaching their maximum permissible values. If x_u/d is greater than $x_{u,max}/d$, then the section has to be redesigned, either by change in the grade of concrete or steel, or by revising the size of section. This is also clearly mentioned in clause G.1.1 (d) of annex G of the new code as well as in the 1978 version, and there is no anomaly in this regard.

(ii) The paper makes a general statement that authors of text books in this country have misinterpreted the codal provisions, and there is a perception among educators and professionals that all reinforced concrete (RC) structures ultimately fail by crushing of concrete. These statements are not true for all cases. Two important publications by the Bureau of Indian Standards (BIS), New Delhi, namely, SP:24-1983, *The explanatory hand-book on IS 456 : 1978,* and SP:16-1980, *Design aids to IS 456 : 1978,* which are widely used by many design engineers in the country, are very clear in their explanations and perceptions about the limit-state design. The tables in SP:16-1980 are also prepared for values up to maximum limiting percentage of tensile reinforcement only. The misinterpretations mentioned about flexural design in the point of view are not warranted, and these are well covered in these two BIS publications. If some authors and educators wrongly interpret the codal provisions, the code is not to be blamed for that.

(iii) The strain diagrams shown in *Fig* 3 of the paper are not as per the new code. The limiting values of strain are 0.0035 and $0.87f_y/E_s + 0.002$ for concrete and steel, respectively, and not as shown in *Fig* 3. Under the balanced condition, that is, when both these strains reach the maximum values, 'x_u' will be '$x_{u,max}$' as given in the code. The strain in the inner layer of tension steel will be linear and proportionately less, and the stress in this inner layer steel can be worked out with respect to the strain.

(iv) The author advocates the use of under-reinforced columns in building frames, and cites the case of bridge piers as an example. This example of bridge piers is not appropriate as these are generally designed as per the code of Indian Road Congress, IRC:21, for road bridges, and railway standards for rail bridges, and not as per IS codes any way. Besides, most of the piers for road bridges in our country are over sized as compared to those provided abroad, for reasons best known only to engineers of the public works department. These cannot be compared to the case of columns in multi-storey buildings. It is agreed that the concept of keeping the axial load close to the balanced point, as shown in *Fig* 2 of the paper, is valid for earthquake resistant structures to avoid brittle compression failure, and not for other types of structures, and for buildings with shear walls. However, for earthquake resistant structures, closely spaced links are to be provided in columns at the joints of columns and beams, as recommended in IS 13920 : 1993 to avoid ductile failure.

(v) The clause on the new code on enhanced shear strength in sections near support is based on the well-established theory of trussed-beam and several research works conducted on the subject worldwide. However, this clause is not to be seen in isolation, but with other requirements of minimum shear reinforcement, etc. Besides, it has to satisfy the requirement mentioned at the end of clause 40.5.1 that the tension reinforcement should extend on each side of the point where it is intersected by a possible failure plane for a distance at least equal to the effective depth, or be provided with an equivalent anchorage. Again, for earthquake

resistant structures, the requirements of shear reinforcement in beams are different, and the recommendations of IS 13920 : 1993 are to be followed.

(vi) Regarding walls, the new code clearly states in clause 32.1 that the walls subjected to axial load and bending moment are to be designed as per Section 5 or Annex B for vertical reinforcement in each face, and not just for axial and shear strengths only as the paper puts it.

(vii) The clauses in the new code on reinforcement detailing of joints for most structures, other than the earthquake resistant type, are well covered, but the designer has to follow it thoroughly to put it effective in practice. For anchorage of tension bars in column-beam joints, the provisions of clause 20.6.2.2.5 of the new code regarding bearing stresses at bends, are to be followed strictly. Many design engineers ignore this requirement in practice for which the code is not to be blamed. In this regard, reference may be made to the explanations to clause 25.2.2.5 of IS 456 : 1978, given in SP:24-1980.

(viii) By and large, the new code covers most of the design aspects for conventional type of framed structures. Nevertheless, some of the points raised in the paper are valid for design and detailing of earthquake-resistant structures, which are very much lacking in the present IS 456 : 2000.

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The author replies:

At the outset, I would like to thank Mr Prabhakar for patiently reading the paper and summarising his impressions. I am personally grateful to him for bringing out some more points through his discussion note.

The following is a point-wise reply of the discussions raised by Mr Prabhakar.

(i) Clause 38.1 of IS 456 : 2000 does not say that RC sections have to be designed as under-reinforced. Prescription for rectangular sections is only suggested through the clauses in the Annex G. It would be of immense value if the code could say explicitly that all sections need to be under-reinforced without leaving it to the designers to surmise from the expressions for specific rectangular sections. This will help those designers who prepare their own computer programs for design of other sections like triangular, T-shaped and L-shaped.

The code does not specify $0.002+0.87f_y/E_s$ as maximum strain in steel. It specifies it as the minimum value for the maximum strain in steel. This means that the bottom row of steel could have a strain larger than this. The question then is "should the inner most row of longitudinal steel have the strain of $0.002+0.87f_y/E_s$ specified as the minimum value?". This is not clarified in the code.

(ii) If SP:16 does not say that the failure is due to crushing of concrete, then why does it do all calculations with the strain in concrete as 0.0035, when it is clear that the strain in concrete cannot reach that value if the section is under-reinforced? The code only states that 0.0035 is the maximum strain in concrete, but it is the design aids handbook and textbooks in the country that have taken it as the mandatory value of strain to be used in the calculation of limiting values of flexural strength.

The code does not make a statement on the philosophy of limit state design that tensile failure is required. The tables in design aids handbook are only suggestive, and the Annex G of IS 456 : 2000 merely touches on one special case. It is the responsibility of the authors of codes and textbooks to give the correct interpretation of the clauses in codes.

(iii) Again, the code specifies $0.002+0.87f_y/E_s$ as the minimum value for the maximum strain in steel. The values of $x_{u,max}$ are based on this. This means that one can have x_u less than $x_{u,max}$, particularly if a strain of 0.0035 has to be reached in under-reinforced section as per the code. This means that the strain in the lower layer of steel could be larger than $0.002 + 0.87f_y/E_s$, which is acceptable by the code. There is no statement in the code that says that the inner layer of tension steel should have proportionally less strain.

(iv) The discussion is valid *even* for large columns of tall RC frame buildings with section sizes of 1.2 m-1.5 m and multiple layers of steel.

It is interesting to read that in structures designed for forces other than seismic and in buildings with shear walls, brittle compression failure is acceptable. The miniscule column sizes adopted in the Indian building frames without regard to the ultimate behaviour of the structure, will force the section to be well above balanced point. Such sections highly under-utilise the steel reinforcement in the sections. This is just the current Indian practice, but the preferred behaviour is still ductile failure with enough warning.

It is surprising to read that the ductility in earthquake-resistant structures is achieved very simply by providing "links" in the beam column joints. This is not correct. It may be noted that links in beam-column joints alone will not assure ductility in RC structures.

(v) While clause 40.5.1 talks about the placing of shear reinforcement, the current clauses for calculation of the area of shear reinforcement is still open-ended with the value of a_v left free to be chosen by the designer. This is unsafe design procedure.

(vi) Clause 32.3.1 suggests neglecting bending moment in the design of walls, if the cross-section is completely in compression. This is a major concern.

(vii) The code does not have any provision for checking the safety of the joint shear stresses arising out of the loading on the structure. The issue of anchorage of longitudinal beam bars does not solve the concern of the joint design.

(viii) The paper refers to seismic design of RC sections as a matter of parallel. However, the issues raised

are relevant for non-seismic design itself first, and then of course for seismic design.

Complacence amongst practising engineers that the recent revision of the code after 22 years without any changes in the earlier clauses on structural design provisions covers the design aspects well, is indeed disheartening.

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The Guest Editor summarises:

The limit state design and the working stress design methods of IS 456 : 2000 are exactly same as of the 1978 version, except for some detailing aspect. The designs are now influenced by the exposure and fire rating considerations. Even though the design method has not changed between the two versions, the final design sections need not be same because of the exposure conditions.

Reinforcement will yield first in under-reinforced concrete structure. The percentage of elongation of mild steel is about 24 and that of high yield strength deformed (HYSD) bars is about 15 with the result that fracture of steel bars is not normally affected before the fracture of the concrete. The percentage of crushing strain of concrete is only 0.35 percent. Invariably, the crushing in concrete strain is reached before the fracture of the steel. This type of failure is called secondary compression failure. It is not compression failure but secondary compression failure.

IS 456 : 2000 does not explicitly state that the beams be designed as under-reinforced concrete. However, the restrictions and the method of design, the minimum strain limitation on reinforcement implies, that the section designed, as per the code, will be an under-reinforced one. The acceptable minimum strain in reinforcement at the centre of gravity of steel specified by the code is the proof strain with a small partial safety factor. One may argue that the steel will not yield at the specified strain. But the partial safety factor applied to concrete is higher than that of the steel. Therefore, the steel will yield before concrete reaches the crushing strain at the limit of strength even in balanced designed. The books, in general, follow the code so the recommendation that the sections be designed as under-reinforced ones is correct.

In under-reinforced concrete beams, the reinforcement yields first. The steel that is used in the present construction is ductile. Even though the failure is initiated by the yielding of steel, concrete reaches the crushing strain faster than the fracture strain of steel. The collapse of the beam will be by crushing of concrete. This is called secondary compression failure and it is different from the primary compression failure. Ultimately, the RC beams collapse by crushing of concrete unless there is reasonable compression steel.

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