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Shortcomings in structural design provisions of IS 456 : 2000

C. V. R. Murty

The fourth revision of the Indian standard code of practice for plain and reinforced concrete, IS 456 has made significant headway in the provisions related to concrete engineering. However, it is observed that most of the provisions related to reinforced concrete (RC) design engineering that were present in the third revision remain untouched. Even the few provisions that were added, namely, those related to design of RC walls and shear design of beams, seem to be inconsistent with the basic understanding of structural behaviour. The provisions regarding design of RC walls do not mention the need for the axial load-bending moment interaction to be considered in their design; which could be considered as a conceptual error introduced in the new revision. The provision on design of shear in beams is based on a narrow perspective of structures designed to resist only static loads. This point of view shows how the new code in the current form can lead to non-conservative designs. Therefore, a comprehensive revision of the RC design engineering provisions is urgently required.

IS 456 provides the basic specifications for design of concrete structures in the country. The code also forms the basis for many other Indian standards related to concrete structures. It has primarily two parts to it; the first part deals with concrete engineering and the second with structural design engineering. A shift in the philosophy of design of reinforced concrete (RC) from the elastic method of working stress method to the in elastic method of limit state method was the most significant change in the third revision of IS 456, hereinafter referred to as 'the old code'[1]. After 22 years of the release of that document, significant changes have been made in the provisions related to concrete engineering in the latest revision of IS 456, hereinafter referred to as 'the new code'[2]. Interestingly, however, the new code offers no changes in provisions related to structural design engineering, barring the addition of one provision on shear design and another on design of RC walls.

The author feels that even after 22 years, the new code has not recognised some of the significant advances made in the RC design across the world. It is only giving a general reference to the detailing aspect of RC member of IS 13920-1993 that was developed on the subject of reinforced concrete. Interestingly, even though the title of the code IS 13920-1993 says it is meant for detailing of RC structures resisting seismic loading, there are clauses in it that are neither prescriptive nor empirical, but are quantitative and based on sound concepts of structural behaviour. IS 13902-1993 uses the concept of capacity design in arriving at the amount of shear reinforcement. This is not merely a matter of detailing. The main thrust in RC design is to be able to produce the desired structural behaviour by suitably providing the reinforcement. None of the revisions of IS 456 have ensured that the desirable modes of failure

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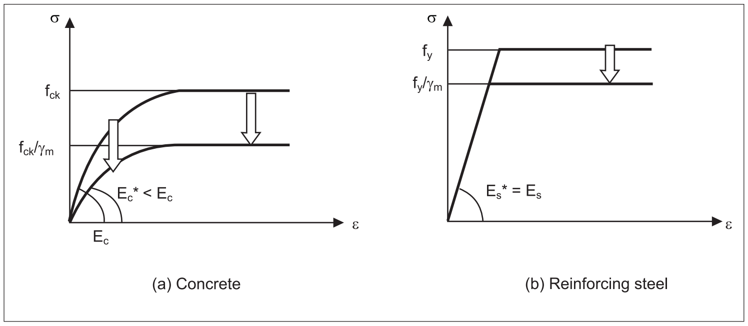

Fig 1 Differences in the application of partial safety factor, γ_m, on concrete and reinforcing steel in-so-far as the initial modulus of elasticity is concerned

(for example, ductile, under-reinforced, flexural failure) occur before the undesirable ones (for example, brittle shear failure). This proper sequencing of ultimate behaviour of RC structures/members is indeed a higher level of design engineering. Of course, structures of under-design loads, other than due to seismic effects, are not expected to go into their ultimate state. But, under seismic effects, they are pushed into their ultimate state and hence, sequencing their failure modes is essential. Clearly, IS : 13920 -1993 employs a higher level of design engineering pertaining to the design of RC members than that presented in the new code. This point of view highlights some major concerns related to safety of structures designed as per the new code.

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Major drawbacks

The fourth revision of the code after 22 years has not shown any changes in the provisions related to structural design. It seems that the existing provisions have neither been properly investigated nor their merits and demerits understood. The following review of some of the provisions shows how precariously supported is the Indian RC design practice.

Material properties and associated factors

The limit state method (LSM) for flexure design in the code gives six underlying assumptions as a basis for estimating the flexural moment capacity of a section. These are given below.

(*i*) Plane sections normal to axis remain plane after bending. This implies that the strain distribution is linear at any cross-section, and that shear deformations are possible as the plane sections are not restricted to remain normal to the axis of bending even after bending.

(*ii*) The stress-strain curve of concrete is specified to be consisting of a parabolic softening branch followed by a plateau. The partial safety factor, γ_m, for material strength is 1.5 for concrete.

(*iii*) The stress-strain curve of reinforcing steel is specified to be bilinear elastic perfectly plastic. The partial safety factor, γ_m, for material is 1.15 for reinforcing steel.

(*iv*) The tension capacity of concrete is neglected.

(*v*) The limiting strain of concrete in compression is 0.0035, and

(*vi*) The minimum value of the limiting strain of reinforcing steel in tension is $0.002 + \left(0.87\, f_y / E_s\right)$

The purpose of applying the partial safety factor, γ_m, for materials in considerations *(ii)* and *(iii)* above is only to account for unreliable strength in the material. This also means that a lower grade material is expected. Owing to the metallurgy of steel, even a lower grade steel has the same initial modulus of elasticity. However, a lower grade concrete has indeed a lower modulus of elasticity. Based on this argument, the code lowers the entire stress-strain curve of concrete, whereby even the modulus of elasticity of concrete, E_c, is also reduced in comparison to the actual (short-term) modulus of elasticity of concrete, E_c, *Fig* 1, and the stress-strain curve of reinforcing steel is lowered such that only the strength is reduced and not the initial modulus of elasticity, E_s, *Fig* 1. However, it is not clear as to whether the modulus of elasticity estimated by $E_c = 5000\sqrt{f_{ck}}$ as per clause 6.2.3.1 of the new code, already takes into account this reduction in the modulus of elasticity of concrete or not; thus, both the calculation of member deflections and the estimation of overall structure drift are under a cloud.

Further, in assumption *(iii)* above, the strain-hardening of steel that is present in all reinforcing steel is not reflected in the idealised stress-strain curve of steel. Hence, the capacity of the member is underestimated. While this error in the strength may not be an important issue in non-seismic structures, it may result in unsafe structures or in structures with undesirable behaviour when designed to resist earthquake forces. This point will be clarified later while discussing the capacity design concept. Such an approach of considering strain hardening of steel in the design stress-strain curves is already in practice in the Euro code[4].

An important feature of concrete as a material is that its strength can be enhanced by providing confinement to it in a direction transverse to that of the compression loading. That is, with the same material:

- a higher quality of concrete can be achieved owing to higher strength, higher maximum strain and reduced cracking, and
- the performance of confined concrete under ultimate load conditions, particularly with respect to its shear behaviour and cracking, is superior to that of concrete without the confinement. The new code does not even mention this aspect of concrete, let alone the quantitative effects of confinement. There are numerous benefits of confining concrete and the code does not take advantage of it.

The new code has formally launched the use of high strength concrete. Experimental studies had shown that the ultimate strain sustained by concrete in compression is smaller for higher grades of concrete. The footnote under *Table* 2 of the new code states that for compressive strengths higher than M *55*, the design parameters given in the standard may not be applicable; one is required to refer to specialised literature to identify these parameters. However, it is established that even for concretes of grades below M 55, there could be a drop of upto 17 percent in the ultimate strain of concrete from the code specified value of 0.0035[5]. Such a drop in the ultimate strain can significantly reduce the ultimate load-carrying capacity of the structure. The code does not recognise this. Fixing the ultimate strain in concrete is a code statement and cannot be left to the discretion of designers, who may choose different values depending on the literature referred to. It is therefore necessary for the code to specify another value of the ultimate strain of concrete, at least for the higher grades of concrete, if not make the ultimate strain a function of the grade of concrete.

Flexure design based on LSM

Based on the assumptions listed in clause 38.1 of the new code, flexure design can be conducted as per the LSM. Interestingly, the code does not clearly emphasise that only under-reinforced sections are to be designed. However, it is suggested only indirectly through the clause G-1.1(d) in Annex G, of the new code that over -reinforced sections are not permitted in beams. The concern is with respect to the ultimate strain ε_{cu} considered in the extreme fibre in concrete, and not with respect to the guaranteed minimum strain in steel. The code embeds the need for designing under-reinforced structures only through formulae/expressions, the importance of which may not be clear to the designer. Authors of textbooks in this country have misinterpreted this unsaid statement. The use of "$0.36 f_{ck}$" as the average stress in concrete in Annex G and the further use of "x_u" from force balance (that is, $0.36 f_{ck} b x_u = 0.87 f_y A_{st}$)has prompted designers /textbook writers to take ε_{cu} to be 0.0035. There is a perception among educators and professionals that all RC structures ultimately fail by the crushing of concrete. But, the LSM of design as conceived in IS 456 requires calculation of ultimate moment when any of the limiting states is reached. Interestingly, an upper limit of 0.0035 is specified for the extreme fibre strain in concrete while a lower limit of $0.002 + (0.87 f_y / E_s)$ for the extreme layer strain in steel. Since the upper limit of strain in steel is two orders of magnitude higher than that of concrete (0.0035), it is conceivable that an under-reinforced structure may keep accumulating strain in tension steel well beyond $0.002 + (0.87 f_y / E_s)$ until eventually the extreme fibre of concrete reaches a strain of 0.0035 and crushes. But, the associated curvatures required to generate this strain of 0.0035 in concrete can be very large and not even realisable, particularly in deeper girders of greater depth. In RC girders that are integrally built with the structural frame, these curvatures required cannot be realised. As a measure to overcome the above difficulty, the code may consider specifying an upper limit for the strain in the extreme layer steel or an upper limit for the maximum curvatures that can be generated in RC beams.

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The code does not make a statement encouraging the design of column sections as under-reinforced, even though the consequences of designing over-reinforced sections are more serious in columns than in beams. With reference to the *P-M* interaction diagram, the design point on the compression axial load side should be made to lie at or below the balanced point *Fig 2*, that is, $P_{design} < P_b$, so that the failure of the column is by yielding of steel and not by crushing of

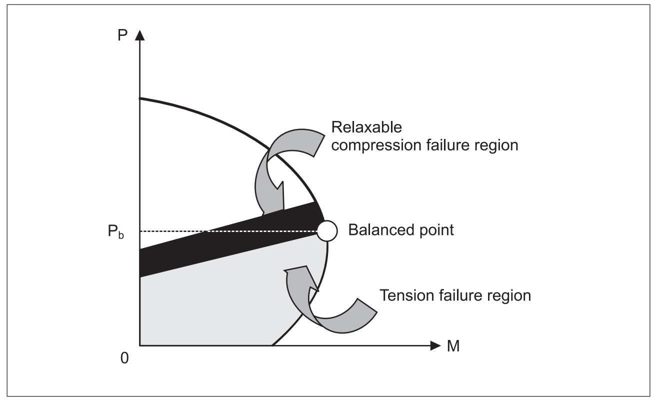

Fig 2 Desirable tension failure region for design of all important columns, and permissible relaxation of this region in columns of less important structures.

concrete, *Fig 2*. It is possible to avoid over-reinforced column sections, particularly when they are subjected to both axial load and bending moment. Bridge piers subjected to lateral forces are examples wherein the area of cross-section is large and the amount of reinforcement is small (usually around one percent), and the design point is, in most cases, below the balanced point. In buildings, the redundancy is higher and the bending moment due to lateral loads per column may be small. Further, the efforts of designers to reduce column sizes to increase architectural appeal pushes the design point more towards the apex of the *P-M* interaction diagram. This is not desirable owing to possible brittle compression failure. Also, smaller column sizes relative to that of the beams suggest that the girders are likely to be stronger than columns. Under lateral loads, this *strong-beam-weak-column* system leads to catastrophic storey collapse mechanisms (or sway mechanisms). From these points of view, it may be required to peg down the design point to a level marginally above the balanced axial load, if not at or below the balanced axial load.

This requirement is essential in structures with low redundancy, for example, bridge piers. In less important structures, this restriction may be relaxed to some extent, *Fig* 2. A practical limit may be placed on the maximum design compressive stress that can be developed in the reinforcement bars; this stress can be, say, 90 percent of the design strength of $0.87 f_y$.

The procedure mentioned in Annex G for the calculation of moment capacity is only for beams even though the code does not mention it. Further, it is applicable only for beams with a single layer of tension steel. Often, beams designed to carry large loads have tension steel in more than one layer. In addition, there are instances of deep beams where significant side face reinforcement may be provided. The designer can take advantage of these reinforcement bars in the ultimate moment capacity of the beam. However, if the procedure given in Annex G is applied to calculate the ultimate moment capacity of such beams, the ultimate moment capacity is over— estimated anywhere from 10 percent to 50 percent depending on the location of the inner layers of steel and the area of tension steel in those layers. Not only is it sufficient to look at the stress in the tension steel, but also necessary to ensure that the strain states being considered in the cross-section are indeed practical. Annex G does not spell out whether the inner layer of steel should also reach the minimum strain of $0.002+\left(0.87 f_y/E_s\right)$. If it has to, then the beam has to undergo very high curvature which may be impractical to achieve. The use of the expressions available in Annex G for obtaining the moment of resistance of the section with layered steel will also lead to large over-estimates of the moment of resistance.

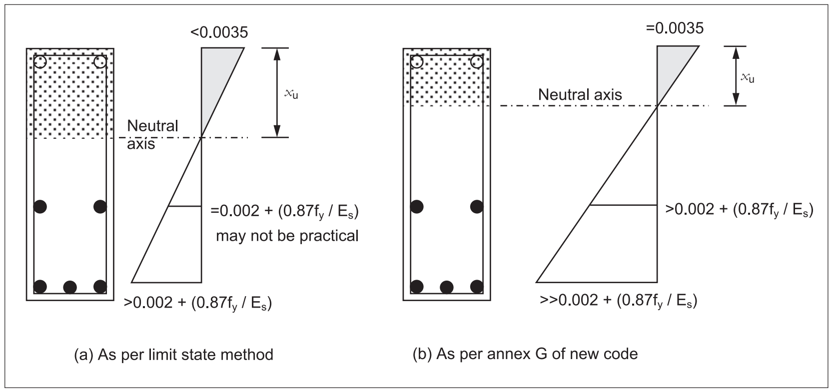

Fig 3 Strain distribution in an under-reinforced beam with two layers of steel with (a) limiting state as intended by the limit state method of flexure design given in IS 456 : 2000, and (b) limiting state as suggested by Annex G of the new code.

The reason for the above over- estimation is explained here. The entire premise of LSM for flexure design is the reaching of the specified limiting strains of 0.0035 by concrete and of $0.002+\left(0.87 f_y/E_s\right)$ by steel, and not the reaching of the stresses corresponding to these limiting strains, namely an average maximum stress of $0.36 f_{ck}$ in concrete and $0.87 f_y$ of steel. The main concern with the procedure given in Annex G is that it determines the depth, x_u, of neutral axis based on the force equilibrium when both steel and concrete reach their limiting design strengths, that is, concrete reaches the maximum average design stress of $0.36 f_{ck}$ and steel of $0.87 f_y$. However, both concrete and steel reach their limiting strengths simultaneously only in balanced sections. As shown in *Fig* 3, this condition does not represent the limiting state of under-reinforced beams. In under-reinforced beams, the concrete does not reach the limiting strain of 0.0035 and only steel reaches the limiting strain of $0.002+\left(0.87 f_y/E_s\right)$. For this reason, the flexural curvature developed in the section, if the calculations are performed as suggested in Annex G, is significantly larger than that obtained if the calculations are performed based on the intended strain-based limiting state. In deep beams, these large curvatures may not even be realised in the section, and inner layers of tension steel may not even reach yield.

The procedure detailed in Annex G is not applicable to large size columns used as bridge piers and to reinforced concrete walls, in which the steel is distributed along the cross-section. Here, invoking clause 38.1(f) requires that the maximum strain in tension reinforcement in the section at failure shall not be less than $0.002+\left(0.87 f_y/E_s\right)$ at the innermost layer of tension steel and ensuring that the outermost layer does not rupture the ultimate moment of resistance, M_u, can be evaluated. However, it is necessary to see if the ultimate curvature so visualised is practical or not. Again, Annex G cannot be used for this purpose.

The indiscriminate use of the procedure suggested in Annex G is also possible in the calculation of the deflections as per Annex C and crack width as per Annex F of the new code. Here, these calculations require the depth of neutral axis x of the section at working load levels. Since the procedure in Annex G does not clarify that the expression for depth x_u of neutral axis as given in clause G-1.1(a) is to be used *only* to evaluate if the section is under-reinforced, balanced or over-reinforced, and *not* for any other calculations (including calculation of ultimate moment of resistance and of deflection), it is feared that designers may mistake this value itself to be the actual neutral axis at the limit state. Unfortunately, the error of using the above x_u in the calculation of ultimate moment of resistance and of deflection is consistently made in noted textbooks, currently in use in the country[6].

Owing to this major conflict in the basic method of calculating the depth x_u of neutral axis in LSM, the following procedure is presented for its calculation; all quantities involved therein in this calculation are noted in *Fig* 4. The grades of concrete and steel are denoted as f_{ck} and f_y, respectively. The calculation of x_u involves setting up a classic problem of structural mechanics by casting all the three sets of equations, namely, of force equilibrium, material constitutive law and strain compatibility.

Force equilibrium equation

$$f_{c,ave}\, b x_u + f_{sc} A_{sc} = f_{st1} A_{st1} + f_{st2} A_{st2} \qquad (1)$$

where,

$f_{c,ave}$ = average stress in concrete

b = width of the compression face

x_u = depth of neutral axis

f_{sc} = stress in compression reinforcement

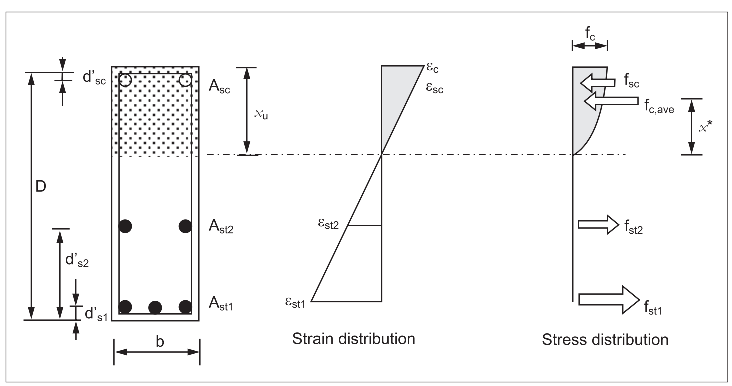

Fig 4 Geometry, strain states and stress states of concrete and steel reinforcement bars in a reinforced concrete section.

A_{sc} = area of compression reinforcement

f_{st1} = Stress in tensile reinforcement in layer 1

A_{st1} = area of tensile reinforcement in layer 1

f_{st2} = Stress in tensile reinforcement layer 2

A_{st2} = area of tensile reinforcement in layer 2

Material constitutive laws

$$
\begin{aligned}
f_{c,ave} &= g(\in_c) \\
f_{sc} &= h(\varepsilon_{sc}) \\
f_{st1} &= h(\varepsilon_{st1}) \\
f_{st2} &= h(\varepsilon_{st2})
\end{aligned}
\qquad (2)
$$

where,

ε_c = limiting strain of concrete
ε_{sc} = limiting strain of compressive reinforcement
ε_{st1} = limiting strain of tensile reinforcement in layer 1

Strain compatibility equations

$$
\begin{aligned}
\varepsilon_c &= \left(\frac{x_u}{D - d'_{s1} - x_u}\right)\varepsilon_{st1} \\
\varepsilon_{sc} &= \left(\frac{x_u - d'_c}{D - d'_{s1} - x_u}\right)\varepsilon_{st1} \\
\varepsilon_{st2} &= \left(\frac{D - d'_{s2} - x_u}{D - d'_{s1} - x_u}\right)\varepsilon_{st1}
\end{aligned}
\qquad (3)
$$

where

D = full depth of beam,

d'_{s1} = effective cover to tensile reinforcement in layer 1

d'_{s2} = effective cover to tensile reinforcement in layer 2, and

d'_c = effective cover to compression reiforcement.

In Eqn.(2), the relation between the strain ε_c in the extreme fibre of concrete and the average stress $f_{c,ave}$ in concrete compression region is given by

$$
f_{c,ave} = \begin{cases} 0.45 f_{ck}\left[\left(\dfrac{\varepsilon_c}{0.0020}\right) - \dfrac{1}{3}\left(\dfrac{\varepsilon_c}{0.0020}\right)^2\right] \\ \quad \text{for } 0 < \varepsilon_c < 0.0020 \\ 0.45 f_{ck}\left[1 - \dfrac{1}{3}\left(\dfrac{\varepsilon_c}{0.0020}\right)\right] \\ \quad \text{for } 0.0020 \le \varepsilon_c < 0.0035 \end{cases} \qquad (4)
$$

where,

f_{ck} = characteristic compressive strength of concrete

In Eqn.(2), the functions $g(\varepsilon_c)$ and $h(\varepsilon_s)$ are the design stress-strain curves of concrete and steel as specified by the code in clauses 38.1*(c)* and 38.1*(e)*, respectively. In the eight equations listed under Eqn.*(1)* to *(3)*, there are nine unknowns, namely, x_u, ε_c, ε_{sc}, ε_{st1}, ε_{st2}, $f_{c,ave}$, f_{sc}, f_{st1} and f_{st2}. Therefore, one equation is short to solve for the nine unknowns. Here, one of the limiting strain values needs to be imposed. For instance, in under-reinforced sections, the limiting strain ε_{st1} of $0.002 + (0.87 f_y / E_s)$ in the tension steel is specified by the code, and, in over-reinforced sections, the limiting strain ε_c of 0.0035 in the extreme concrete fibre in compression is specified. This enables the solution of all the unknowns. Owing to the non-linear nature of the functions $g(\varepsilon_c)$ and $h(\varepsilon_s)$ an iterative process becomes necessary. Thus, the depth x_u of neutral axis is obtained.

After obtaining x_u, the ultimate moment of resistance M_u is obtained as :

$$
\begin{aligned}
M_u = {} & f_{c,ave} b x_u (x^*) + f_{sc} A_{sc} (x_u - d'_{sc}) + \\
& f_{st1} A_{st1} (D - d'_{st1} - x_u) + f_{st2} A_{st2} (D - d'_{st2} - x_u)
\end{aligned}
\qquad (5)
$$

where,

$$
x^* = \begin{cases} \dfrac{\left[\dfrac{2}{3}\left(\dfrac{\varepsilon_c}{0.0020}\right)^2 - \dfrac{1}{4}\left(\dfrac{\varepsilon_c}{0.0020}\right)^3\right]}{\left[\left(\dfrac{\varepsilon_c}{0.0020}\right) - \dfrac{1}{3}\left(\dfrac{\varepsilon_c}{0.0020}\right)^2\right]} x_u & \text{for } 0 < \varepsilon_c < 0.0020 \\ \dfrac{\left[\dfrac{1}{2} - \dfrac{1}{12}\left(\dfrac{\varepsilon_c}{0.0020}\right)^2\right]}{\left[1 - \dfrac{1}{3}\left(\dfrac{\varepsilon_c}{0.0020}\right)\right]} x_u & \text{for } 0.0020 \le \varepsilon_c < 0.0035 \end{cases} \qquad (6)
$$

In summary, Annex G is only for design of a restricted class of RC rectangular sections. Its indiscriminate use may result in highly non-conservative structures. In particular, the method cannot be used to obtain the ultimate capacity of a section whose properties (of geometry, materials and reinforcement bars) are known, that is, to review the capacity of a known section.

Shear design based on LSM

Traditional wisdom from experiments say that shear behaviour is wanting in RC structures; failure in shear is sudden and brittle. In contrast, the behaviour of under-reinforced flexure members is gradual and ductile. Therefore, it is widely accepted that shear design must always be conservative, so that flexure design governs the behaviour of the structure. Interestingly, this wisdom seems to have eluded the new code. Through clause 40.5 of the new code, a downward revision is sought in the shear reinforcement in beams close to the support, under the pretext of enhanced shear strength in that region. The basic safety traded in by reducing the shear reinforcement and increasing the vulnerability to shear failure is not acceptable. Also, in comparison to the total reinforcement in the structure, the reduction in the quantity of shear reinforcement achieved through this clause is marginal, even to compel its use from economic considerations. This clause is particularly non-conservative for structures to be built in earthquake-prone areas, wherein shear failure is to be delayed to force the ductile flexure failure to occur first.

It is unfortunate that the new code includes such retrograde provisions that are detrimental to the safety of the structure. Such dangerous modifications of shear design provisions near supports where the shear force is critical, seem to have emerged from a shortsighted point of view of design of structures to resist only static loads. Further, clause 40.5 also seeks to increase the shear strength of concrete $(2d\tau_c/a_v)$ near the supports upto the maximum shear stress $\tau_{c,\max}$. This implies that there could be situations where the shear force at the critical section is large, but only minimum shear reinforcement is provided, owing to this peculiar provision in the new code. The clause also seeks to provide shear reinforcement along the length of the beam when the section to be considered is very close to the support and not along the direction transverse to the beam. It would have been educative to the designer if the new code could have also provided detailing schemes to support this requirement. Since a designer does not know the failure plane angle, it is a mystery as to how one will identify a_v to be used in the suggested new shear calculations. It is surprising that the code has such a weakly conceived provision. This non-conservative shear design provision should be urgently withdrawn by the Bureau of Indian Standards.

Experimental and analytical studies have shown that shear stresses are the most dominant in the behaviour of RC frame joints[7]. It is necessary to urgently develop appropriate design provisions for design of RC joints for inclusion in the IS 456

Design of RC beam-column joints

The new code deals only with the design of components and not of the system. In this approach, the code overlooks an important feature in the design of connections between the various components. In particular, since RC moment-resisting frames constitute most of the building construction in the country, the absence of guidelines for the design of frame joints is conspicuous. In the absence of formal guidelines in the Indian Standards, the designer is free to use international literature and design those elements or components of the structure. However, code provisions mean more than mere calculations. Code provisions are a statement of the risk that the country/community is willing to take while building the structures. Therefore, even though it is possible to borrow provisions from different codes and complete the design of the frame, it is evident that the risk in each of these elements will not be consistent.

The success of the design of RC frames lies in ensuring the safe transfer of forces from the beams to the columns. Experimental and analytical studies have shown that shear stresses are the most dominant in the behaviour of RC frame joints[7]. There are two issues in the design of joints, namely the anchorage of beam bars into the column and the control of shear stresses in the joint region. Detailed treatment of this subject of the design of RC frame joints from the IS 456 context is just available[8]. It is necessary to urgently develop appropriate design provisions for design of RC joints for inclusion in the IS 456.

Seismic design of RC structures – Capacity design concept

The new code provides partial safety factors for combining all loads, including the seismic forces. Even before the publication of the new code, a separate Indian standard was already available for the design of RC structures subjected to seismic forces, which covers the design and detailing of RC structures based on advanced concepts of seismic design[3]. Inspite of this, the new code does not even mention this and fails to refer the designers to use that document for developing earthquake-resistant structures. Thus, the new code also fails in informing designers of the latest developments related to the design of RC structures.

New concepts in the design of RC structures have evolved through the improved understanding of the behaviour and design of earthquake-resistant construction. The earthquake-resistant design philosophy is different from the design philosophy for all other forces acting on structures, either natural or man-made. Earthquake-resistant structures are deliberately designed for a much smaller force than that would be developed if they were to remain elastic. In other words, earthquake-resistant structures are designed to resist strong ground shaking by large inelastic actions so that the input seismic energy can be dissipated. Therefore, an earthquake-resistant structure is required

- to possess good ability to absorb seismic energy through inelastic deformations, and
- to develop a favourable collapse mechanism in the event of strong earthquake shaking.

Thus, adequate ductility must be built into the structure, and a hierarchy must be formed amongst the various failure modes by virtue of which the ductile failure *modes* occur before the brittle ones. Ductility of a section is enhanced through proper confinement of concrete, while that of the structure is ensured by the development of a suitable collapse mechanism which distributes the inelasticity uniformly throughout the structure at many different locations, enhancing overall energy dissipation potential and limiting the inelastic demand at each of these locations to a low level. These locations of inelastic activity are to be so detailed that they can undergo large deformations before losing strength and stiffness.

This is achieved by employing the capacity design concept[9]. This concept provides a calculation procedure for ensuring that the flexure failure modes occur before the shear failure modes. This is done by estimating the lower bound shear force demand on the structure from the overstrength plastic moment capacities developed at critical sections in the possible collapse mechanism of the frame. The overstrength plastic moment capacity of the frame section is obtained by the procedure described in section 2.2 for obtaining the ultimate moment of resistance with the partial safety factor for concrete of 1.5 and for steel of 1.15 replaced by 1.0 and 0.8, respectively. The factor 0.8 for steel accounts for strain-hardening in reinforcement bars. A detailed treatment of this well-established calculation procedure in the context of IS 456 is given elsewhere. The concept is already in use in IS:13920-1993, but is not referred to in the new code.

In the design of RC sections, all brittle modes of failure are to be avoided. The new code specifies a design procedure that requires the designers to perform the flexure design first, and then perform the shear design. Here, the percentage p_t of longitudinal flexural tensile steel is used as an input in the estimation of shear strength τ_c of concrete required in shear design of the section. However, no mechanism is embedded in the code by which the shear failure is delayed to occur after the flexural failure. Thus, it is not guaranteed that the section will behave in a ductile manner, even though it is designed to behave in an under-reinforced manner in so far as flexure is concerned. Moreover, in the design of earthquake-resistant structures, the importance of a provision to ensure that flexural failure occurs before shear failure cannot be over-emphasised.

Limit state design of RC walls

The new code has more provisions for the design of RC walls, than the old one. The provisions require independent checks for the axial strength and the shear strength. Interestingly, these provisions in the new code seem to suggest that RC walls are not related to bending moment capacities. This implies that nothing can be said about the flexural strength of RC wall structures designed by the new code, once again questioning the safety of the structure.

The design of beams for flexure needs to be re-considered particularly the way it is treated in Annex G of the new code... The design of columns requires to be constrained such that the design point lies primarily in the tension failure region of the interaction diagram

An RC wall section is like a column with uniformly distributed longitudinal steel in it. Thus, the *P-M* interaction that is noted in columns is also valid for these walls. A detailed calculation procedure is required to obtain the *P-M* interaction surfaces. The procedure discussed through Eqn.(1) to (6) is required to be extended by including the axial load P in Eqn.(1). While doing so, the limiting strain for the section in compression, as listed in clause 39.1(*a*) and (*b*) in the new code, are to be kept in mind. For RC columns, a limit needs to be specified on the axial load P in RC walls also to be less than that around the balanced axial load P_b, or on the stresses in the compression reinforcement to be not less than 80 percent of the design strength $0.87 f_y$ to ensure that the failure mode is a ductile one and not of sudden type.

Design of infilled RC frames

Almost all RC frames built in India have masonry infills. Currently, the design practice is to neglect the presence of infills and assume the entire load to be carried by the bare frame. However, the infills contribute significantly to the strength and stiffness of the structure. Analytical studies and experiments have shown that there are large beneficial effects of considering infills in the design of the structure[11,12,13]. The new code provides no guidance to designers on how to include the same in the design. Such provisions are already available in the codes of other countries[14,15]. If the designed structure is close to the actual one, then the structure constructed will behave as expected; else, damage will be imperative. Detailed guidelines for the design of infilled RC frames are necessary in the Indian code to encourage the design and construction of efficient and economic structures.

Provisions on effective length of frame members

The design tool given in the current code in Annex E to estimate effective lengths of frame columns is too simplistic and based on assumptions that may not even be realised under actual conditions. If actual conditions of geometry, stiffness, restraints and loading of the structure deviate from those assumed in deriving it, then it may not even give reasonable results. This tool given in Annex E is over three decades old. Since then, a large amount of research has been conducted in this area; an excellent summary of the same is available[16]. Codes of many countries have already taken advantage of the same and modified their provisions accordingly. The method of calculations of effective lengths in the Indian codes need to be urgently modernised to cover more realistic frame columns.

Closing remarks

The fourth revision of IS 456 : 2000 does not include any major changes in structural design provisions. There are a few additions in the provisions for the design of RC walls and to explain the enhanced shear strength near the supports in beams. Both these provisions are based on very weak premises. There is an urgent need to hold off these provisions, particularly the one related to design of shear near the supports. The design of beams for flexure needs to be re-considered particularly the way it is treated in Annex G of the new code, while demonstrating the method of implementing the limit state design procedure for flexure. Further, the design of columns requires to be constrained such that the design point lies primarily in the tension failure region of the interaction diagram. Detailed provisions for an design of RC frame joints are an immediate necessity so as to design and construct efficient and effective RC moment-resisting frame structures.

More modern design concepts like the capacity design concept need to be brought into the code. Guidelines to ensure a desirable failure hierarchy, whereby shear failure is delayed beyond the flexure failure, are required to increase the quality of RC design engineering. It is urged that the Bureau of Indian Standards immediately takes up a comprehensive review of the RC design engineering provisions of the code and remove the conceptual errors, shortcomings and inconsistencies present in it and integrate it with the other available Indian standard codes of practice related to RC design of structures.

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