Discussion Forum

Design of flexural members subjected to axial force

I read with interest the Point of View titled "Design of flexural members subjected to axial force" by Mr M.C. Upadhyay, published in the April 2004 issue of the journal[1]. I would like to congratulate the author for developing an *MS-Excel* program for designing the flexural member section with unequal reinforcement to arrive at an economical solution, as compared to providing equal reinforcement at both faces as per SP 16:1980[2]. However, I would like to discuss below some of the points concerning 'unsafe design' as per SP 16 which is mentioned in the write-up.

To prove his point on an economical design, the author has given an example of a rectangular section subjected to axial compressive force and bending moment. By his program, the total steel area required is 1046 mm^2, of which compression steel area is 155 mm^2 and tension steel area is 891 mm^2, as against a higher steel area of 1500 mm^2, that is 750 mm^2 at both faces, as per SP 16. His point may be accepted considering the fact that there is no reversal of moment on the section and the value of $P_u / f_{ck}bD$ in the example is only 0.0667 which is less than 0.1, and bending moment governs the design.

Further, the author states that since tensile steel area of 750 mm^2 as per SP 16 is less than the tensile steel area of 891 mm^2 calculated by him, the design using SP 16 tends to be "unsafe" on the tension side. This is a wrong comparison as the steel areas and placement of bars in the section in each method are different by which design using SP 16 cannot be called as unsafe. The steel area obtained by using SP 16 is compatible with stress and strain relations and the applied loads, and this has been checked by a rigorous method using *MS-Excel* spreadsheet *RECTCOLUMN.xls*[3]. Hence, the use of SP 16 for the example considered is in order and safe, although it is not economical in terms of total steel area.

It is well known that in the design of member sections by limit state method, for a safe design, many combinations of steel areas and arrangement are possible for a given load, each of which providing different neutral-axis depth and stress-strain values, but all these satisfying the codal limitations on stress and strain. The most ideal and economical solutions are arrived at when the strains of concrete and steel reach the maximum value simultaneously for the applied load considered, that is termed as the balance design. For the example considered in the paper, this balanced condition exists when there is no compression steel (as the compressive load is very small) and the tensile steel area is equal to 1010 mm^2, which is more than the tensile steel area of 891 mm^2 calculated by the author. By the same analogy put forward by the author, it cannot be said that the section designed by the author is unsafe on the tension side.

The paper states that as per SP 16, for the points below line $fst = fyd$, the outermost tension reinforcement undergoes inelastic deformation, that is, the stress in the reinforcement is more than the design yield strength ($0.87fy$). The latter part of this statement is not correct as the stress will continue to be $0.87fy$ only even after the tension reinforcement undergoes a certain degree of inelastic deformation before the concrete fails in compression under limit state of collapse, which is a ductile failure. This is clearly evident from the stress-strain curves for steel and concrete given in SP 16 and IS456:2000[4]. In the limit state design for flexure, it is usually the under-reinforced section as defined in SP 16 that is preferred as it results in the ductile failure mentioned above, instead of a brittle failure. In fact, IS456:-2000 stipulates for flexural members that the maximum strain in tension reinforcement at failure shall not be less than $f_y / 1.15\ E_s + 0.002$, by which it means that the steel strain can be anything more as the concrete eventually fails in compression under the limit state of collapse. For this reason, there is no need to have a constraint $E_{st} = 0.0038$ for *Fe* 415 as indicated in flow chart *Fig* 2 of the paper.

I trust that the readers of the journal will find the above observations useful.

Mr. N. Prabhakar
Structural Consultant
Flat E-8 Star Residency,
Evershine City, Vasai (E) 401208
Thane District

References

1. UPADHYAY, M.C. Design of flexural members subjected to axial force, *The Indian Concrete Journal,* April 2004, Vol.78, No.4, pp 53-54.
2. ______*Design aids for reinforced concrete to IS 456:1978*, IS SP 16: Bureau of Indian Standards, New Delhi.
3. ______MS-Excel Spreadsheet *RECTCOLUMN.xls* developed by N. Prabhakar, *www.sefindia.org* of Structural Engineers Forum of India.
4. ______*Indian standard code of practice for plain and reinforced concrete*, IS456:2000, Bureau of Indian Standards, New Delhi.

The author replies:

I thank the discusser for his interest in my write-up. I have deliberately chosen the example of a section which is subjected to a moment (150 kNm) less than the balanced moment of resistance (167.7 kNm). If we choose to design it ignoring the axial compressive force that is only for moment we will get no compression reinforcement as it is an under-reinforced section. If we design it for axial compression too with moment, it is shown in the write-up that it requires compression reinforcement and it certainly reduces tensile reinforcement as compared to the area obtained for only moment condition.

If I increase the force so that the axial stress becomes $0.1f_{ck}$ keeping the same moment (moment 150 kNm and axial compression 300kN), it requires 298 mm^2 area as compression reinforcement. So if I design the section for only moment, I may provide only 2ϕ12 bars (on compression side as binding bars). Certainly this area (226 mm^2) is less than the required (298 mm^2).So there is no alternate to exact design. This is just a small example. The variations may be high depending upon problem to problem. If the applied moment is more than the balanced moment capacity of the section (that is the concrete has reached to maximum compressive stress) then any amount of compressive force needs compressive reinforcement.

And if the beam is subjected to axial tension, more tensile area will be required as compared to the area required only for moment.

I would like to state that the actual stress will be more than 0.87 f_y. Our IS 456 stress strain curve for steel is an idealised curve. There is no clear yield point for high strength steel (that is *Fe* 415; 500) that is why we consider 0.2 percent proof stress as yield stress as the stress strain curve does not become horizontal; it is always increasing, that is, stress increases as strain increases.

The constraint on steel strain is to arrive at the solution for the steel area. The strain will be equal to or more than 0.0038 but the stress will be constant as per the idealised stress strain curve of IS 456 hence the tensile force has maximum value at this strain.

Mr Mukesh Chandra Upadhyay
Senior Design Engineer
J. P. Ventures Ltd. (Consultancy
Organisation of Jaiprakash Ind. Ltd.)
64/4, Site 4, Industrial Area, Sahibabad
Ghaziabad 201010

• • •

Design of a simply supported one-way slab loaded by a strip load along the span

This has reference to the paper titled "Design of simply supported one-way slab loaded by a strip load along the span" which was published in the February 2004 issue, Vol 78, No 2, pp. 117-118 of the *Journal*.

I would like to ask the author whether we should also determine the effective width for the strip load because we have to provide reinforcement in that width only.

Unless that is done how would we determine the effective width for the strip load?

Mr Mukesh Chandra Upadhyay
Senior Design Engineer
M.E. (Str. Engg.) B.E. (Civil)
J. P. Ventures Ltd. (Consultancy
Organisation of Jaiprakash Ind. Ltd.)
64/4, Site 4, Industrial Area, Sahibabad
Ghaziabad 201010

The author replies:

I would like to thank the discusser for going through my paper carefully.

As regards his query, please refer to the basic formula given in clause 24.3.2.1 of IS 456: 2000 which gives the effective width of slab b_{ef}, in terms of the distance of the centroid of the concentrated load from the nearer support, x. When a one-way slab is loaded by a strip load along its span (a case for which I have derived a closed form solution in my paper), the width within which the reinforcement is to be provided would be the width calculated by the formula in Clause 24.3.2.1 of IS 456: 2000 where x is substituted by $l_{ef}/2$ (midspan).

Further, if the designer wishes to curtail the reinforcement which he has provided, he can do it after ensuring that at the structural location of curtailment (that is, location beyond which the curtailed reinforcement is not required), the balance reinforcement is within the width b_{ef} calculated corresponding to that location (that is, x = distance of that point from the nearer support).

I hope this answers the dicusser's query. Please feel free to seek further clarifications, if any.

Mr Umesh Dhargalkar
Director
Dhargalkar Tehnoesis (I) Pvt. Ltd
C-210, Grace Plazqa
S.V. Road
Jogeshwari (West)
Mumbai 400 102

• • •