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Research Article

DESIGN AIDS FOR SERVICEABILITY

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ABSTRACT
Concrete, perhaps, is the most popular and versatile construction material used in wide range and variety of structures. The mouldability of concrete to any desired shape and form is the most attractive proposition to the architect and engineer to build structures, functional and monumental. Compatibility of concrete with other materials has further increased its use in composite constructions too. The bi-strength material property and inherent deficiency of cracking are the causes for concern in the use of concrete. Though, several valid analysis and design assumptions have been made, used and adopted, the problem of serviceability and durability persist and haunt the designer. The alarmingly high rate of non-performance including several catastrophic collapses have shattered the notion that concrete is maintenance and problem free. Research has indicated that embedded steel corrodes faster than exposed steel, and has aggravated designer's concern. An attempt is made to address the problem of serviceability considering crack and deflection control. This work presents quick and efficient design aids for serviceability.

Key Words: *Design Aids, Crack Control, Deflection Control.*

INTRODUCTION

Concrete is unquestionably, the most important civil engineering construction material. The strength properties of concrete are importance. Compressive strength of concrete is very high and it increases with age, if moisture in concrete is prevented from drying up.

Good concrete is relatively durable, under a wide variety of exposures. With strong dense aggregates, a suitably low permeability can be achieved by having a sufficient low water cement ratio. Also with through compaction and sufficient hydration of cement through proper curing, highly durable concrete can be obtained.

Concrete appears to be the most versatile construction material, for, it can be molded into any shape and form. Also because, it is the most compatible material that can be used at congested and crowded sites of limited access. Concrete can be pumped through to such sites, by alerting its workability with a little high water-cement ratio.

The versatility of concrete, the wide availability of its component materials, the unique ease of shaping its form to meet strength and functional requirements, together with the exciting potential of further improvements and developments of reinforced concrete construction, combine to make concrete strong and competitive construction material.

Cracking

Engineers have had a wrong lotion for that, the reinforced concrete maintenance free. However, a large number of failures of concrete structures, of late, have urged the engineers to think otherwise. It has been learnt that, most of concrete failures have been due to corrosion of reinforced steel. Also, it has been found that, the rate of corrosion of embedded steel is faster than that of exposed steel.

Cracks do not, per se, indicate a lack of serviceability or durability. However, it should be ensured that with an adequate probability, cracks will not impair the serviceability and durability of the structure. Because, large scale cracks tend to harm the indented lifetime or the utility period of the structure. This is because, the resultant ingress of moisture thorough cracks, eventually lead to corrosion of the reinforcement.

Deflection

By having deflections as one of the controlling criteria for design, we have an added advantage, since creep and shrinkage effects are better accounted for as deformations, rather than stresses. Through, control of deflections is emphasized, it is very important to know that, control on deflections is one of the many criterions for design and in majority of cases the criterion of strength or criterion of limited crack width governs the design.

However, in certain cases like radar platforms, individual floors on which high precision machineries would be installed, and pavements, deflection criterion takes the driver seat in design. These are certain cases in which one cannot afford to have excessive deflections and rigorous deflection calculations would be prompted.

PERTINENT LITERATURE

Extensive investigations have been carried out concerning crack widths in reinforced concrete members.

Cracking

A paper in a proceeding (Levi, 1961), has devolved a formula for crack width on theoretical and empirical grounds. However, this is based on the assumption of a certain distribution of bound stresses along the reinforced and of uniform or linear distribution of tensile stresses within the concrete tensile zone. Analysis of the stress distribution in cracked reinforced concrete reinforced concrete members has indicated that the thickness of the concrete cover or the spacing of the individual reinforcing bars.

A paper in a proceeding (Broms, 1965), presents a simple method for the calculation of cracks width and cracks spacing on the basis of calculated and measured stress distributions.

A paper in a proceeding (Nawy, 1972), has suggested methods of cracks control for bundled bars.

A paper in a proceeding (Lutz, 1974), present a work, which enables handling of bundled bars and bars of different sizes in a logical manner. Specifically, a very simple conclusion is reached for the equivalent number of bars to be used.

A paper in a proceeding (Nawy, 1972), proposes that magnitude of fracture is a function of the energy absorbed per specific surface of reinforcement. Hence, a proper choice of reinforcement grid size can control cracking in to preferred orthogonal grids of narrow cracks rather than random, widely spaced, diagonal wide cracks. He also states that, since two-way acting slabs are subjected to redistribution of stresses in their fields, an average stress level in excess of 40 % of the yield strength of the reinforcement can result in the development of unacceptable crack width.

While the need for defining a crack limited state of design has been felt, the formula for prediction of crack width varies widely in the codes of practice of various countries. Since the width of cracks that occur in the structural elements that are actually in use have a larger scatter with respect to location and the prevailing environmental conditions, it is not feasible to search for the maximum crack width. Instead, the characteristic crack width between 90% confidence limits is more pragmatic and useful. Some of the recognized approaches of crack width calculation in code include CEB-FIP approach, Australian and British approach.

The (CEB-FIP, 1994), approach is the most versatile of the crack width prediction approaches currently available and is equally valid for prestressed and non-prestressed concrete. It can also be used to predict crack width in shear.

Both BS: 8110-1985 and Australian approaches are toward a non-slip hypothesis considering the very small permissible crack width, and the crack width very close to bar is neglected.

The Australian code AS: 1480-1974 specifies that the flexural crack width used not to be checked under normal exposure conditions if the clear distance between bars does not exceed 350, 220 or 200 mm for f_y=230, 410 or 450 N/mm^2 respectively. Otherwise the crack width under service load must be checked and small be limited to 0.3 mm for conditions of moderate exposure and limited 0.004 times the nominal cover to main reinforcement for serve exposure.

The IS: 456-2000, code provisions are the same as Australian code. Unfortunately Indian code too fails alike a few other codes in specifying something concrete about crack width and crack spacing calculation.

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It just specifies a standard for limiting crack width and does not dwell in to the subtleties involved with the reinforcement detailing versus crack width.

The ACI: 318-1977, approach is worth an elaborate mention, since present work is based on this fracture hypothesis.

Deflection

There has been a lot of research even in the field of deflection over past few decades, the deflection under service loads of a continuous reinforced concrete beam or simply supported beam must use the actual moment pattern in a member. The moment pattern, in the case of continuous beams is a function of the flexural rigidity EI, which is affected by the cracking across the span of the member.

An HPR report (Branson, 1963), proposed design procedures for computing deflections and states that heavily reinforced members, the uncracked transformed section can more accurately be used instead of the gross section Ig. He also defines an expression for the average Moment of inertia over the entire length of a simply supported beam of rectangular or T-sections.

A paper in a journal (Jain , 1972), in their work, come up with the simple procedures for computing short-time and long-time deflections of reinforced concrete members. Here, for short-time deflection a general formula is prescribed. For long-time deflection, an equation is proposed which takes into account most of the factors affecting the same. Charts and tables are given to facilitate the computations of deflections.

Deflection control: different codal recommendations

The IS: 456-2000, states that, the vertical deflection limits may generally be assumed to be satisfied provided that the span to effective depth ratios are not greater than, 7 for cantilevers, 20 for simply supported, and 26 for continuous. As known to all, deflection is end product of loading process. Hence, the criterion of limiting deflection based on span to depth seems to be illogical. Neither the span nor the depth is the direct reflection of loading pattern.

The special publication 16 (SP-16), proposes ready envelops with maximum span to depth ratio versus percentage tensile reinforcement for deflection control. Furthermore, the expression suggested for Ieff in the calculation of deflection, is highly tedious and its validity is open to question. Furthermore, such an elaborate equation is uncalled for in quick deflection calculations.

This has prompted us to conduct a detailed study on the serviceability of reinforced concrete and an attempt has been made to provide an integrated approach for the design of reinforced concrete structures based both on strength and serviceability. In such an endeavor design for reinforced concrete beams has been proposed based on limiting crack width and deflection criteria.

THEORY AND DEVELOPMENT OF DESIGN AIDS FOR SERVICEABILITY

General

The main serviceabilities of reinforced concrete, namely, cracking and deflection have been dealt by many researchers. There has been a lot of research on crack propagation, crack width, crack spacing, crack pattern etc. Similarly, a number of research minds have also dealt deflection of reinforced concrete numbers. Hundreds of formulae have been proposed for the calculation of deflection.

Cracking

The ACI code approach to limit the crack width is based on the fracture hypothesis by Gergley and Lutz. The maximum crack width formula at the level of reinforcement is given by,

$$\omega_m = 10.996\ x 10^{-6}\ \beta\ f_s (Adc)^{1/3}\ mm \text{, with usual notations} \quad (1)$$

The ACI code suggests that, the factor $f_s(Adc)^{1/3}$ x 10^{-3} be limited to 25 kN/m and 30 kN/m for exterior and interior exposures respectively. This is analogous to liming maximum crack width to 0.33 mm and 0.41 mm for respective exposures. This shows that, though the expression for crack width is realistic in ACI code, the approach toward limitation of crack width is almost similar to other codes.

From the equation above, it can be observed that, the factors affecting the crack width are, Grade of steel used, Area of tensile zone and Effective cover to tensile reinforcement. The first factor itself suggests us that, the control of crack width becomes more critical in higher grades of steel like Fe415 and Fe500, rather than mild steel. Furthermore, it should be observed that, with the aim of preventing corrosion of reinforcement bars, the cover to reinforcement should not be indiscriminately increased, since crack width is directly proportional to effective cover an area of tensile zone. Hence, an optimum value of effective cover has to be selected based on limiting crack width.

The following equations are derived for crack width based on reinforcement scheduling for beam sections,

$$\omega_m = 2.51\, x\, 10^{-6}\, \frac{M}{A_b}\, n^{-4/3} \qquad (2)$$

Where M is the service moment in kNm, n- is number of reinforcement bars and A_b – is the area of reinforcement bar.

$$\omega_m = 2.41\, x\, 10^{-6}\, \frac{M}{A_b}\, n^{-4/3} \qquad (3)$$

In the equations (2) and (3), we can observe that, with known service moment, number of particular diameter of bar to be provided, to limit the crack width to a required value can be calculated.

From the equations, it can be seen that crack width varies to the negative power of 4/3 with number of reinforcement bars. In other words, more the number of bar, less is the crack width. From this it leads us to infer that, with more circumferential area of contact of reinforcement bars with concrete, crack width can be minimized.

As an example, we shall consider that, we have to provide reinforcement area of 1200 mm^2, for a beam. We could be satisfying the area requirement by providing #4 of 20Φ. An alternate bar selection with essentially the same area, #6 of 16 Φ can also be provided. However, the perimeter of both the detailing methods yields different values. The former arrangement provides a perimeter of 80 π, whereas the latter provides a perimeter of 96 π and obviously latter arrangement is better to the former. Hence, the formula should help the designer in choosing a particular mode of scheduling in preference to the other equally practical modes; yielding higher value of perimeter to limit the crack width within required limits.

Using equations (2) and (3) charts are prepared for both the grades of steel, and in each grade of steel, for each available bar diameter in market. Having known the service moment of resistance and the value to which crack width has to be limited, these charts aid the engineer to obtain number of reinforcement bars of particular diameter. The charts are prepared in such a way, as to provide flexibility for the designer to switch on to other practical modes of detailing, if necessary. This is enabled by including some moment values in more than one chart.

Derivation of basic equation for crack width based of effective depth

We know that, for mild steel reinforcement f_s = 140 N/mm^2, therefore,

$$\omega_m = 304.81\, x\, 10^{-6}\, (1/n)^{1/3}\, d \qquad (4)$$

$$\omega_m = 500.76\, x\, 10^{-6}\, (1/n)^{1/3}\, d \qquad (5)$$

These equations (4) and (5) would aid the engineer in selecting depth, based on liming crack width criterion, for the beam which has been designed already for steel requirement. Using equations (4) and (5) two charts are prepared for each grade of steel namely, Fe250 and Fe415, to obtain effective depth based on limiting crack width and reinforcement scheduling.

These two sets of charts should help the designer, completely design the beam (i.e., to select depth and proper reinforcement detailing) based on limiting crack width criterion.

Deflection

Control of deflections in IS code is achieved, generally by restraining the span to depth ratio to certain values based on the boundary conditions. Further, modifying factors are suggested to account for effects of grade of concrete and steel and percentage of tension and compression reinforcement. This procedure works out to be very tedious and cumbersome with many modification factors coming into play.

However, Sp-16 suggests charts 21, 22 and 23, which control the deflection based on span to depth envelope with percentage of tension reinforcement. Though the charts are a success in integrating all grades of concrete with a grade of steel in one chart, certain modification factors are suggested for length of beam and boundary conditions.

Here is a proposition of improvement of above work. Each grade of steel and concrete and every type boundary conditions practically existent are addressed for, which no modification factors at all.

The basic equation for deflection is given by the equation,

$$\delta = k_1 \frac{M\, l^2}{E_c I} \quad (6)$$

Where, M=Moment of resistance of beam, l= Span of beam, E_c=Young's modulus of elasticity of concrete, I=Moment of Inertia of the beam and k_l=A factor depending on the boundary and loading conditions.

With reference to the control of deflections IS code states that, the final deflection due to all loads including the effects of temperature, creep and shrinkage and measured form the as-cast level of the supports of floors, roofs and all other horizontal members should not normally exceed span by 250.

i.e., deflection ≤ span/250, $\delta \leq 1/250$

Substituting the above in (6) we get,

$$k_1 \frac{M\, l^2}{E_c I} \leq \frac{1}{250} \quad (7)$$

In the deflection calculation of SP-16 we can see that, there are two suggested values of modulus of elasticity 'E' for concrete. One, which is $E_e=5700\sqrt{f_{ck}}$ N/mm^2 where f_{ck} is in N/mm^2, used in deflection calculation due to loads. Second, which is given by, $E_{ce}=E_c/(1+\theta)$, where, θ is creep co-efficient based on age at loading. This value of E is suggested for deflection calculation due to creep effects.

However, since our deflection calculation approach is integrated (i.e., all inclusive due to loading, creep and shrinkage), for all practical purposes, young's modulus of elasticity for concrete can be taken as,

$$E_C = E_S / m = 2\, x\, 10^5 / m \quad (8)$$

$$I_{cracked} = \frac{jk^2}{2} bd^3 \quad (9)$$

Also we know that, $M = \sigma_{st} A_{st}\, j\, d$ (10)

Where, $A_{st} = p \times b \times d /100$, where, p = Percentage of tension reinforcement.

Substituting (8), (9) and (10) in (7),

$$\frac{k_1 \sigma_{st} A_{st}\, jdl}{(jk^2/2)bd^3 \times 2\times10^5/m} \leq \frac{l}{250}\;\; k_1\left[\frac{\sigma_{st} m}{k^2\times10^7}\right] p(l/d) \leq \frac{l}{250} \quad (11)$$

We can see that except 'p' and (1/d) all other factors are constants based on grade of steel, concrete, boundary conditions and loading conditions.

Derivation of charts

Two sets of charts are presented, for two popular grades of steel the first set of charts comprising of C1 to C-19, correspond to the limiting crack width. Here, again C-1 to C-9 has to be employed for Fe 415 (high- tension) grade of steel. Further, the charts C-1 to C-8 and C-10 to C-18 correspond to each bar diameter of steel ranging from 8 Φ to 32 Φ.

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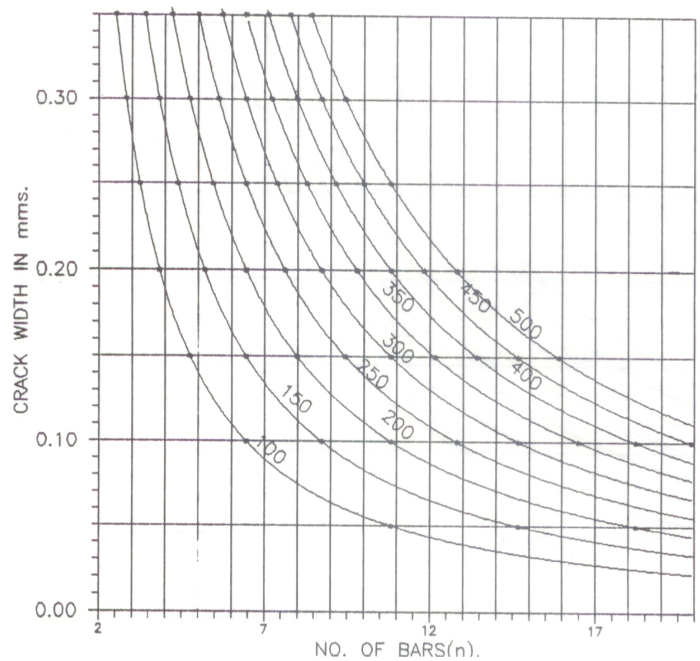

Figure 1: Moment of resistance in kNm, Bar diameter 16 mm, fy =415 N/mm^2

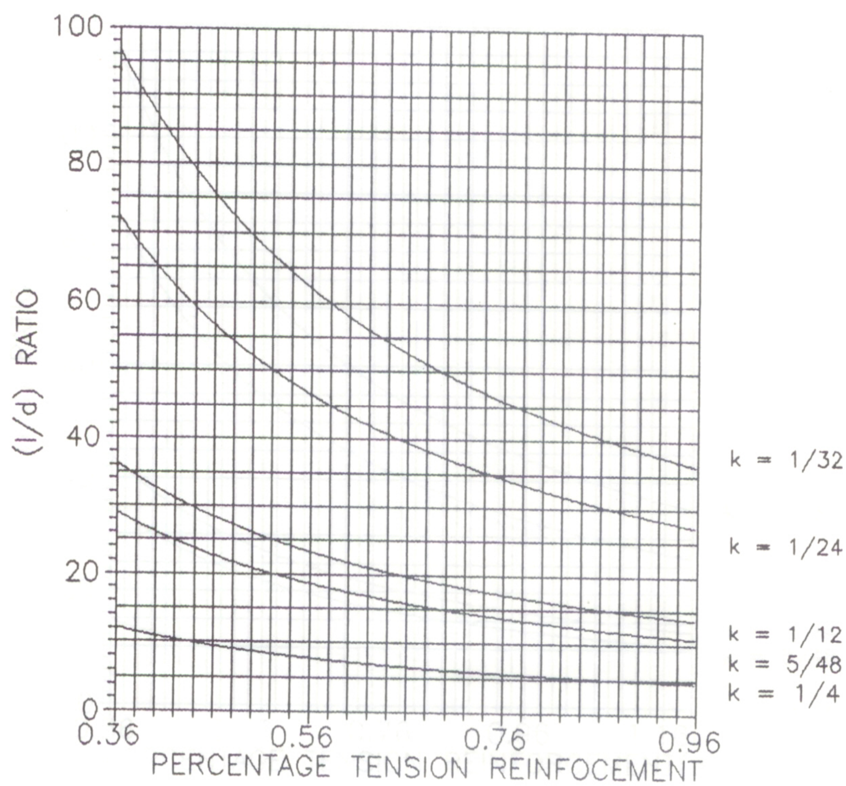

Figure 2: Envelopes for deflection control for fy =415 N/mm^2 and fck =20 N/mm^2

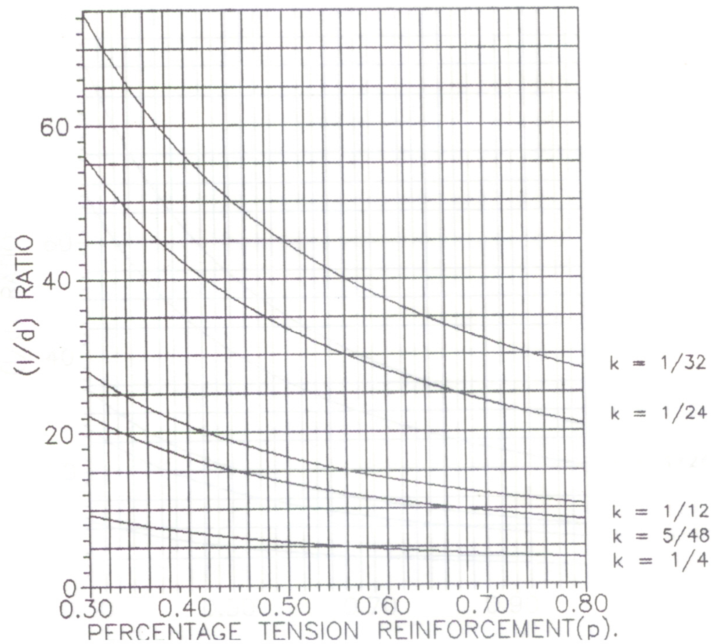

Figure 3: Envelopes for deflection control for fy =500 N/mm^2 and fck =20 N/mm^2

Charts M-1 to M-9 comprise of second set of charts. Here, charts M-1 to M-3 respectively, correspond to deflection control charts for M15, M20 and M25 grade of concrete, with Fe250 grade of steel.
Further, charts M-4 to M-6 respectively, correspond to deflection control charts for M15, M20 and M25 grade of concrete, with Fe415 grade of steel.
Lastly, charts M-7 to M-9 respectively, correspond to deflection control charts for M15, M20 and M25 grade of concrete, with Fe500 grade of steel.
In this paper C13, M5 and M8 charts are presented.

Conclusions

The importance of a check on serviceability cannot be over-emphasized. Durability also heavily relies on serviceability. However, codal recommendations in this regard are either to crude or painfully elaborate, leading to scepticism or for gross rejection. The code feels that, if detailing requirements are satisfied cracking should not be a concern, which need not be the case always. The simplified approach control deflection based on span to depth ratio and its modification bears no relationships with loading pattern. Further, computation of Ieff, long term effects are too lengthy and unwarranted.

The method proposed has elegance, versatility; simplicity, applicability and the shear speed with which assessment can be made should elevate it to the level of valid decision making tool at the designer's office.

The wide range of strengths, grades, bar diameters, loading dispositions and boundary conditions for which design curves have been devised and produced should help he designed in solving serviceability problems of structural elements of a wide range of magnitude and variety, including that of doubly-reinforced sections.

Serviceability aids have been presented keeping in mind, the magnitudes of strengths of elements that are normally addressed, the popular grades of materials generally adopted, the diameters of the steel bars that are popular and readily available, the loading pattern and disposition usually encountered, and boundary conditions for elements that often occur.

The highlight of the approach is strict adherence to all codal requirements including long term effects of material properties on serviceability, which should prompt the designer to accept the method as a convenient method based on logics.
The design aids can also be employed to check the adequacy of the existing steel and reinforcement, of structures in distress.

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