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BEHAVIOR OF SLENDER COLUMN SUBJECTED TO ECCENTRIC LOADING

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ABSTRACT

This paper focuses on Behavior of slender column subjected to eccentric loading. Six slender, reinforced concrete columns with slenderness ratio equals to 15; the compressive strength of the concrete were ranged from 60 to 100 MPa. Slender column were subjected to eccentric axial load with load-eccentricity: depth ratio of 0.15. Three columns were reinforced with six bars having a nominal strength of 415 MPa and other three were reinforced with same number of bars having strength equals to 500 MPa with longitudinal steel ratio equals to 4%. The test results were compared with the values predicted using IS 456-2000. These test, enabled the provision for slender columns in the code to be checked against experimental values, have indicated that IS 456-2000 are very safe and uneconomical design document for HPC slender column.

INTRODUCTION

Increasing demands of the new millennium for sustainable and durable structures, and the limited available resources, have awakened the need for newer construction technologies and efficient use of structural materials. With growing number of tall buildings and mammoth structures the high performance concrete is becoming concrete is becoming more popular. High Performance Concrete (HPC) is defined as concrete that meets special performance and uniformity requirements that cannot always be achieved routinely by using conventional materials and normal mixing, placing and curing practices.

To achieve taller buildings with more rentable areas and also for aesthetic purpose,
high strength reinforced concrete slender columns have become more and more popular in the construction industry. Reinforced concrete slender columns are an important class of structural elements whose behavior and structural responses are not yet well understood. Columns are usually thought of as straight compression members whose lengths are considerably greater than their cross-sectional dimensions.

Despite a large number of investigation carried out in the past on behavior of high
strength concrete (HPC) columns, controversy still remains with regard to some vital design issues. One such issue is the behavior of column with the use of high yield strength steel (Fe415 and Fe500). Six columns are casted and tested eccentrically. The strength of the concrete varies from 60MPa, 80Mpa and 100Mpa. The main parameters is to draw interaction curve, to find the ultimate load and compare with IS 456-2000.

RESEARCH SIGNIFICANCE

This research develops an experimental program that will fill the gap in lack of knowledge regarding behavior of slender column. Moreover, it also intends to close the existing controversy in literature about the behavior of slender column.
The primary objective of the research is to investigate the effect of replacement of high yield strength steel. The proposed research includes testing of columns with the variables such as strength of concrete with 60 MPa to 100 MPa of varying yield strength of steel. The moment interaction curves are drawn for each column and determined the ultimate load and moment which is then compared with IS 456-2000.

EXPERIMENTAL PROGRAM

The experimental program includes casting of cubes of the required strength and then casting of columns of the same strength. The object of this program is to study the behavior of such column under uniaxial bending, to draw the load-moment interaction diagram of different column. This experimental program started with the concept to study the behavior of Long column (slender column) and to examine theavailability of different codes on prediction of ultimate load and moment.Experimentally speaking, it started with the casting of cubes for 60, 80 and 100MPa. Thestrength was achieved after two trials mix. Six column were casted, two were of 60MPa with different grade of steel (Fe415 and Fe500) and the same with 80 and 100MPa. The cross section of the column was 130mm by 200mm, and the length was 1.98m. Theprogram of adopting such section and the length is to create slender effect and theslenderness ratio equals to 15. Longitudinal steel reinforcement were provided 4% of the cross section, the lateral ties were provided 50mm c/c up to L/4 from top and bottom and 100mm c/c for the middle portion.

Experimental Setup Fig.1

MOMENT INTERACTION CURVE

The design of RC columns is more difficult than the design of RC beams. In practice the longitudinal steel in an RC column is usually chosen with the aid of an interaction diagram. An interaction diagram is a graphical summary of the ultimate bending capacity of a range of RC columns with different dimensions and areas of longitudinal reinforcement. The 'interaction curve' is a complete graphical representation of the design strength of a uniaxially eccentrically loaded column of given proportions.

COLUMN IN ECCENTRIC LOADING

Axially loaded columns occur rarely in practice because some bending is almost always present, as evidenced by the slight initial crookedness of columns.The combination of an Axial load and bending moment Mu is equivalent to a load applied at eccentricity.

A rectangular section with bars at two faces,
loaded eccentrically at the ultimate load,appear in Fig. 3. The neutral axis depth is considered to be less than the overall depth.
Tension failure or a compression failure can occur, depending on whether the tension steel reaches the yield strength. However, a compression failure cannot be avoided by limiting the steel area; since the type off failure is dependent on axial load level. Equilibrium conditions require the resultant compressive force $C_c + C_s$ in the section to be equal to and act opposite to and through the point of application of the external load P_o[Fig.3]. Applying the condition of static equilibrium, it follows that the two design
Strength components are easily obtainable as:

$$P_{ur} = C_c + C_s \qquad (4.1)$$

$$M_{ur} = M_c + M_s \qquad (4.2)$$

Where M_c and M_s denote the resultant moments due to C_c and C_s respectively, with respect to the centroidal axis. Generalized expressions for the resultant force in concrete (C_c) as well as its moment (M_c) with respect to the centroidal axis of bending are given as follows,

$$C_c = a f_{ck} bD \qquad (4.3)$$

$$M_c = C_c\left(\frac{D}{2} - \overline{x}\right) \qquad (4.4)$$

where a stress block area factor

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x≡ distance between highly compressed edge and the line of action of C_c(i.e., centroid of stress block area)

$$a = 0.362x_u/D \text{for} x_u \leq D \tag{4.5}$$

$$a = 0.447(1 - \frac{4g}{21}) \text{for} x_u > D \tag{4.6}$$

$$\bar{x} = 0.416x_u \qquad \text{for} x_u \leq D \tag{4.7}$$

$$\bar{x} = \left(0.5 - \frac{8g}{49}\right) D/(1 - \frac{4g}{21}) \text{for} x_u > D \tag{4.8}$$

$$\text{Where } g = 16/(\frac{7x_u}{D} - 3)^2 \tag{4.9}$$

Similarly, the expressions for the resultant force in the steel (C_s) as well as its moment (M_s) with respect to the centroidal axis of bending is given by:

$$C_s = \sum_{i=1}^{n} (f_{si} - f_{ci}) A_{si} \tag{4.10}$$

$$M_s = \sum_{i=1}^{n} (f_{si} - f_{ci}) A_{si} y_i \tag{4.11}$$

Where,

A_{si}=Area of steel in ith row

y_i=distance of ith row of steel from centroidal axis, measured positive in the direction towards the highly compressed edge

f_{si}=Tensile stress in the ith row of steel

ε_{si}=Strain in the ith row

f_{ci}=Compressive stress in the concrete in the ith row

A balanced failure occurs when the tension steel reaches the yield strength and extreme fibre concrete compressive strain reaches 0.003 at the same time. A tension failure occurs if P_{ur}<P_b and compressive failure occurs if P_{ur}>P_b. By referring the strain profile of the column moment interaction curve are developed in excel program

RESULTS AND DISCUSSION

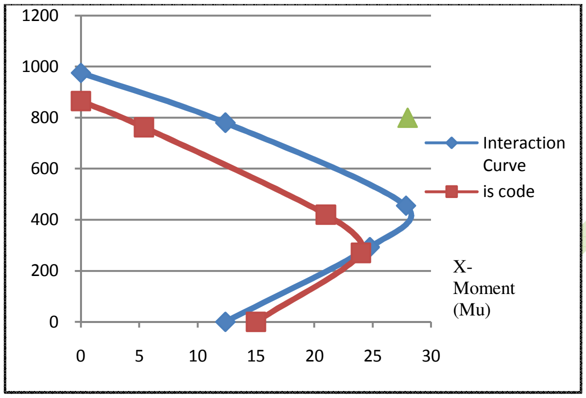

Fig. 6.1 Interaction curve for 60-500

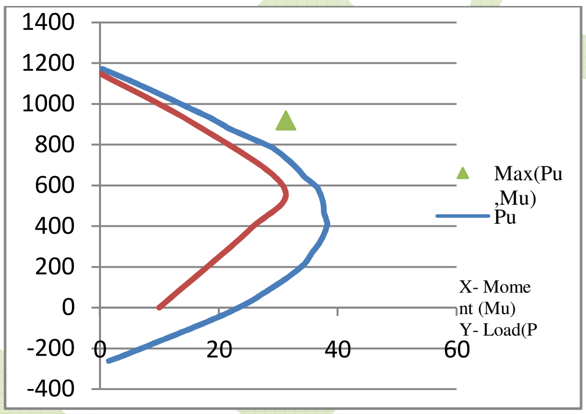

Fig. 6.2 Interaction curve for 80-415

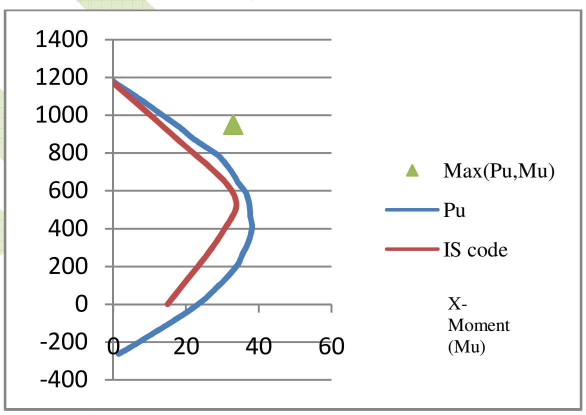

Fig. 6.3 Interaction curve for 80-500

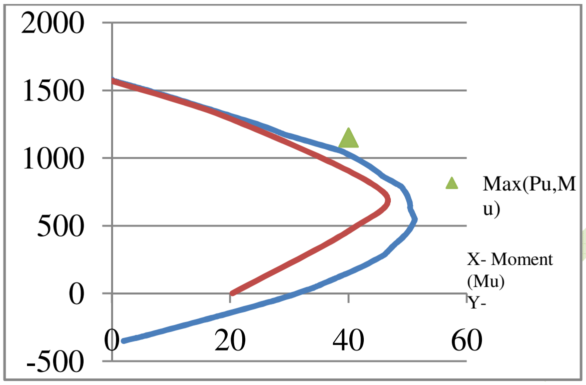

Fig. 6.4 Interaction curve for 100-415

Table 6.1 Comparison of experimental and theoretical results

Column Designation	f_{cu} in MPa	M Test in KN-m	P Test KN	M(IS code) KN-m	P(IS code) KN
60-415	64	***	***	***	***
60-500	65	28	800	18	500
80-415	83	32	850	25	740
80-500	84	34	980	27	750
100-415	105	40	1150	35	1000

The above table 6.1 gives the details of the ultimate experimental load and ultimate load prediction by IS-456-2000. There is very large difference between load prediction by IS-456-2000 and the experimental load and Moment respectively. This provides us that IS-456-2000 is very safer side for HPC and is too conservative towards prediction of load. Whereas experimental results is not too conservative as IS 456-2000 in the light of Moment-Interaction curve and also lies in safer side.

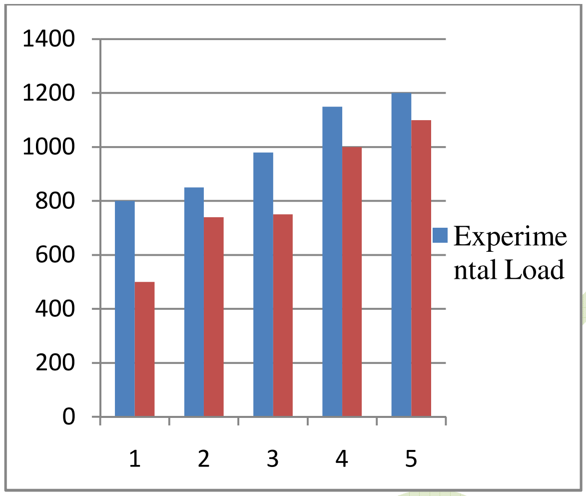

Fig. 6.2 Comparison of Experimental Load and Ultimate load prediction ofcode

This table gives the ratio of experimental load and prediction of load by IS 456-2000. From the ratio we can understand that where does this column lies whether safer zone or danger zone or in between these. The point here to be noted that if ratio is more than 1 it is in safer zone. From table 6.2 results of columns is more than one. The average ratio given by IS 456-2000 is 1.27 whereas the average ratio by experimental results is 1.1. Load and moment prediction of IS 456-2000 is too conservative and this too conservativeness will obstacle the application of HPC column. Experimental results are not too conservative and can be consider for prediction of HPC column in an optimum way.

CONCLUSION

1. HPC column with high strength tends towards brittleness leading less deflection at the mid span.

2. Decreasing the spacing of lateral ties at both the ends up to certain distance has influenced to resist the shear generated due to uniaxial bending.

3. IS 456-2000 code provision is too conservative for the prediction of ultimate load and moment, need to modify for HPC column.

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