

Cyclic Behaviour Of R.C Columns With Various Longitudinal Reinforcement Configurations

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ABSTRACT: In the design of reinforced concrete structures, particularly those for supports of bridges (pier caps, bents, etc), a great deal of reinforcement must be placed in areas where available space is limited. Congestion of reinforcement and difficulty in placing and consolidating concrete often result. One way to solve is to place the reinforcing bars in bundles. Another solution is to arrange the bars in multiple layers. The clear spacing between reinforcement groups will be increased considerably by using bundled bars. Larger spacing will greatly facilitate concrete placement and insertion of spud vibrators and hence improve the quality of the concrete. In the construction of reinforced concrete structures, it is sometimes advantageous or even necessary to place reinforcement in bundles of two, three, or four instead of fixing individual bars at equal spacing. Bundles permit flexibility in detailing where availability of larger bars is restricted and ease manual handling on site. Bundling of bars allows for increased clear spacing, facilitating compaction of concrete between bundles. In this work the effect of bundled bars in R.C columns under lateral cyclic loading is to be studied and comparison with the conventional reinforcement arrangement has to be performed. The cross section of the square column is 450 mm X 450 mm .Scaled down R.C column specimens with two, three and four bars bundled at the corner are subjected to quasi static cyclic loading. From the results obtained the failure pattern, ultimate deflection and load carrying capacity, ductility will be determined. A comparison with the conventional reinforcement arrangement was performed.

KEYWORDS: column, bundled bars.

I.INTRODUCTION

Columns are vertical compression members of a structural frame intended to support the load-carrying beams. They transmit loads from the upper floors to the lower levels and then to the soil through the foundations. Reinforcements are provided in order to reduce the size of the column. The main reinforcement in columns is longitudinal, parallel to the direction of the load and consists of bars arranged in a square, rectangular, or circular shape. The main steel is provided to resist compression load along with concrete. As per Indian standards, a minimum area of reinforcement 0.8 percent and maximum area of 6 percent of the gross cross sectional area of the column shall be provided[6]. The object of stipulating a minimum percentage of steel is to make provision to prevent buckling of column due to any accidental eccentricity of the load on it. For stipulating a maximum percentage of steel is to provide reinforcement with in such a limit to avoid congestion of reinforcement which would make it very difficult to place the concrete and consolidate it. Columns are key structural elements for the seismic performance of buildings. Therefore, special attention should be given to their structural response under load reversals. The collapse of a RC building is caused, in the majority of cases, by the failure of the vertical members. Some of the typical causes of column failure or damage in RC buildings during earthquakes are: (1) lack of stirrups/hoops, confinement and ductility (2) bond/anchorage/lap-slices slipping and bond splitting (3) inadequate shear capacity (4) inadequate flexural capacity (5) strong-beam weak-column mechanisms, etc. Some of these categories are in fact related to design, detailing and construction deficiencies, for which the extensive damage or collapse of structures can occur through the contribution of many of these causes. Columns properly designed for earthquake loading are able to prevent brittle failure and to have a ductile behaviour. In fact adequate reinforcement and detailing is very important in providing the ductile behaviour of RC columns. Over the decades, the design and complexities of reinforced concrete structures have evolved to where concrete structures are designed with ever increasing reinforcing and concrete design requirements. This is especially true in areas of seismic design considerations. American Concrete Institute (ACI) 318 Section 21.2.3 states: "In selecting member sizes for earthquake-resistant structures, it is important to consider problems related to congestion of

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reinforcement. The designer should ensure that all reinforcement can be assembled and placed and that concrete can be cast and consolidated properly. Use of upper limits of reinforcement ratios permitted is likely to lead to insurmountable construction problems especially at frame joints.”[3].

Congested reinforcement can lead to many problems of placing and compacting concrete. In extreme cases this can result in poorly compacted areas into which no concrete has penetrated. Typically heavily congested reinforcement occurs in beam/beam and beam/column junctions and in bridge deck slabs [2]. In the design of reinforced concrete structures, particularly those for supports of bridges (pier caps, bents, etc.), a great deal of reinforcement must be placed in areas where available space is limited. Congestion of reinforcement and difficulty in placing and consolidating concrete often result. One way to solve is to place the reinforcing bars in bundles. Another solution is to arrange the bars in multiple layers. The clear spacing between reinforcement groups will be increased considerably by using bundled bars [1]. Larger spacing will greatly facilitate concrete placement and insertion of spud vibrators and hence improve the quality of the concrete

II. EXPERIMENTAL PROGRAMME

The following sections deals with the details of the experimental programme used in this study.

A. PRELIMINARY INVESTIGATION

The preliminary experimental investigation consists of test on constituent materials and mix proportioning. Cement used in all mixes was Portland Pozzolana Cement conforming to IS specification [4]. Commercially available Manufactured sand passing through 4.75 mm sieve was used as fine aggregate. The physical properties of M-Sand was tested as per IS specifications [5]. Specific gravity and fineness modulus of Manufactured sand used were 2.44 and 2.72 respectively. The size of crushed aggregate used in this test was 12.5 mm. The properties of fine and coarse aggregate conformed to the IS specification [6]. Specific gravity of the coarse aggregate used was 2.72. Potable drinking water available in the college water supply system was used for casting as well for curing of the test specimens. TMT steel rods of yield stress 363 N/mm^2 was used as reinforcement were used for the study. The design of M_{30} mix was done as per IS specification [7]. The properties of all ingredients of concrete were determined and mix proportion was arrived at. After mix design and trial mixes, optimum mix was found as 1:1.41:2.65 with a w/c ratio of 0.44. It yielded a 28 day compressive strength of 36.3 N/mm^2 . The specimens for the study were prepared with optimum mix and cured for 28 days.

B. DESIGN AND DETAILING OF SPECIMENS

The dimensions of the square column specimens used for the test was of 150 mm x 150 mm sides and 500 mm height. The critical column was considered and designed as per IS 456:2000. The specimens are scaled down to one-third its original size. Specimens with bundled bars of two, three and four at corners and non-bundled bars are considered. 4 mm galvanized iron bars are used as lateral ties. The dimensions and reinforcement details of the specimen are shown in Fig.1.

C. CASTING AND TESTING OF SPECIMENS

RCC concrete columns of square cross section were prepared with M_{30} mix. The columns were bundled with one, two, three and four bars at corners as detailed in Table I. Fig.2 shows the schematic diagram of the test setup. The specimens were tested in the 200T capacity self-straining testing frame. Loading was done with push and pull Hydraulic jack with the test set up in the Laboratory. During testing, the observations were made for first crack cycle and ultimate load. Readings for loads in a deflection controlled loading set up were observed and recorded. The displacement controlled loading sequence consisted of drift-controlled mode with three increments of 0.25% drift followed by increments of 0.5% up to 4.5% drift. A metallic collar was inserted to the tip of the column through which the loading was given. A base fixer is fixed at the bottom plate of 200T testing frame. This support condition was chosen to simulate the fixity condition of the column in the field. The loading history of cyclic loading is given in Fig.3.

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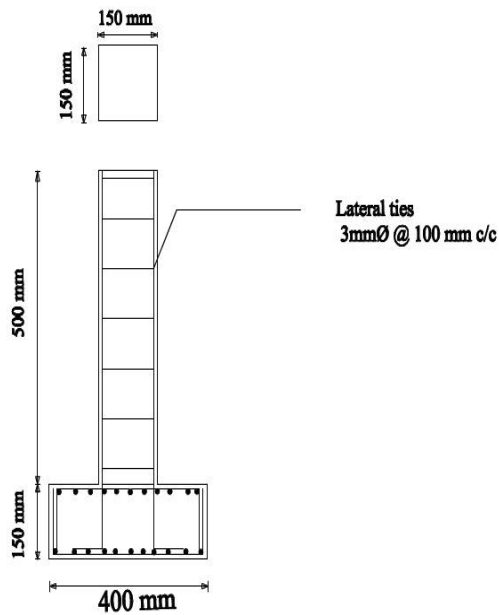


Fig. 1. Reinforcement detailing of column specimens

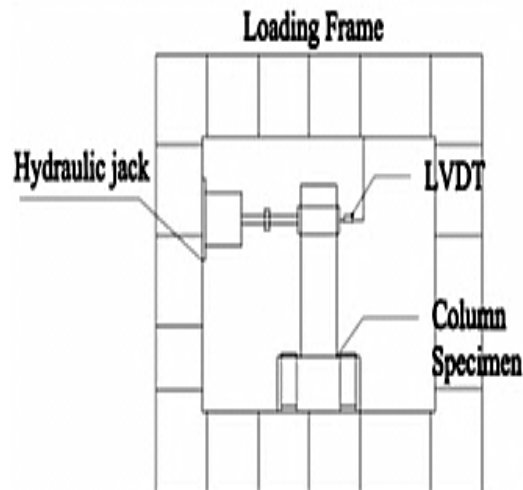


Fig. 2. Experimental test setup

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TABLE I. SPECIMEN DETAILS AND DESIGNATION

Specimen Designation	Details	Dimension (mm)
CB1	Column without bundled bars at corners	150 mm x 150 mm
CB2	Column with two bars bundled at corner	150 mm x 150 mm
CB3	Column with three bars bundled at corner	150 mm x 150 mm
CB4	Column with four bars bundled at corner	150 mm x 150 mm

III. RESULTS AND DISCUSSIONS

The deflection at the top of the cantilever column was taken with LVDT in a deflection controlled loading setup and the load deflection curve was plotted. The test results are represented and compared in terms of load-displacement hysteresis loop, load-deflection envelope, energy dissipation capacity, and ductility and cracking patterns. The observations made during test are discussed.

A. HYSTERESIS CURVE

The comparison of load-displacement hysteresis loops for the specimens are shown in Figure 3. Hysteretic loops show the performance of column under cyclic loading. The area under the curve denotes the potential energy stored in the structure at the maximum displacement position. The Hysteresis behaviour of reinforced concrete is characterized by the rounding and pinching of the loops. The wider the loops, the larger will be the energy dissipation capacity and better will be the seismic performance.

From the hysteresis loop the following behavior was observed:

- Stiffness decreased with the increasing loading cycles due to flexural cracking of concrete and flexural yielding of the longitudinal reinforcement.
- During load reversal the loading stiffness in the next cycle was noticeably lower than the preceding cycle. This could be due to the formation of flexural cracks during the loading cycle.
- The specimen CB1, with conventional reinforcement arrangement, had undergone hysteretic response when it was subjected to the cyclic loading and had capacity to withstand upto 8th cycle with the maximum drift capacity of 3% with maximum energy dissipation capacity at the last cycle to the least energy dissipation at the initial cycle of 65.844 and the ratio of maximum stiffness at the initial cycle to the least stiffness at the last cycle of 6.26, having a uniform increase in energy dissipation with the progressive loading cycles and having better energy dissipation capacity at the initial cycles of loading.
- The specimen CB2, with two bars bundled, had undergone hysteretic response when it was subjected to the cyclic loading and had capacity to withstand upto 7th cycle with the maximum drift capacity of 3% with maximum energy dissipation capacity at the last cycle to the least energy dissipation at the initial cycle of 58.746 and the ratio of maximum stiffness at the initial cycle to the least stiffness at the last cycle of 5.96, having a uniform increase in energy dissipation with the progressive loading cycles and having better energy dissipation capacity at the initial cycles of loading.

- The specimen CB3, with three bars bundled, had undergone hysteretic response when it was subjected to the cyclic loading and had capacity to withstand upto 6th cycle with the maximum drift capacity of 2.5% with maximum energy dissipation capacity at the last cycle to the least energy dissipation at the initial cycle of 54.46 and the ratio of maximum stiffness at the initial cycle to the least stiffness at the last cycle of 5.9.
- The specimen CB4, with four bars bundled, had undergone hysteretic response when it was subjected to the cyclic loading and had capacity to withstand upto 6th cycle with the maximum drift capacity of 2.5% with maximum energy dissipation capacity at the last cycle to the least energy dissipation at the initial cycle of 46.808 and the ratio of maximum stiffness at the initial cycle to the least stiffness at the last cycle of 5.75

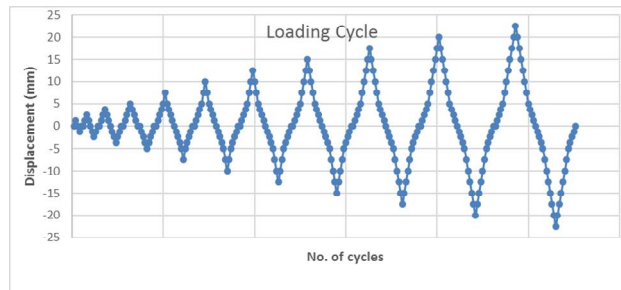


Fig. 3. Loading history of cyclic loading

B. LOAD-DEFLECTION ENVELOPE

The maximum loads and displacements obtained in each half cycle were used for plotting the load displacement envelopes for all tested specimens as shown in Fig.4. The envelope enables the comparison of relative performance of the different specimens

- For columns with conventional reinforcement arrangement, CB1, the stiffness degradation in each cycle was not abrupt and the ultimate failure occurred at 3% drift.
- For columns with two bars bundled, CB2, it followed similar stiffness degradation pattern as that of CB1, and ultimate failure occurred at 2.5% drift.
- For columns with three bars bundled, CB3, there was an abrupt change in stiffness degradation and ultimate failure occurred at 2% drift.
- For columns with four bars bundled, CB4, there was an abrupt change in stiffness degradation than CB3 and ultimate failure occurred at 2% drift.

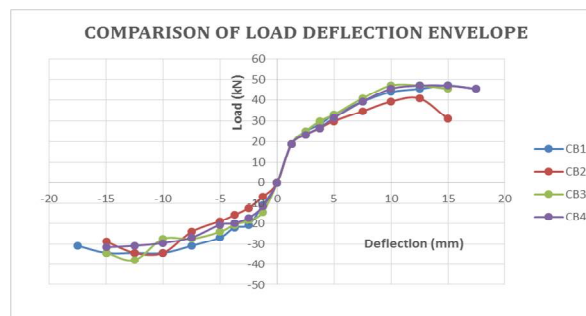


Fig 4: Comparison of load-deflection envelope

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C. Energy dissipation capacity

As a measure of the dissipated energy of the specimens, the area under the full load displacement envelope was computed and defined as the energy that could be dissipated by the specimens before the system lost its stability. Table II summarises the details of energy dissipation capacity of the specimens at yield which in turn has direct effect on the ductile behavior of the specimens.

Specimen	Energy dissipation capacity (kNmm)	% variation
CB1	1346.71	-
CB2	1503.505	11.64
CB3	1477.88	9.74
CB4	1276.038	-5.25

Table II: Energy dissipation capacity of tested specimens

D Yield and Ultimate displacement response

Yield displacement indicates the stiffness characteristics of a structure before yielding. Thus for lower the yield displacement, better will be the stiffness characteristics. The specimen with conventional reinforcement arrangement, CB1 showed lower yield displacement, which indicates higher stiffness. The yield displacement for CB2 was comparable with that of CB1. But for the specimens CB3 and CB4 the yield displacement was higher, which showed poor performance.

With bundling of bars, the load carrying capacity of the specimens was found to be reduced. The ultimate displacement was higher for the specimen CB1 which showed good performance while for the specimen CB2 it was slightly higher than CB1. But for the specimens with three and four bars bundled, CB3 and CB4, there was a reduction in the ultimate displacement. Figure 5 shows comparison of yield and ultimate displacements of testes specimens.

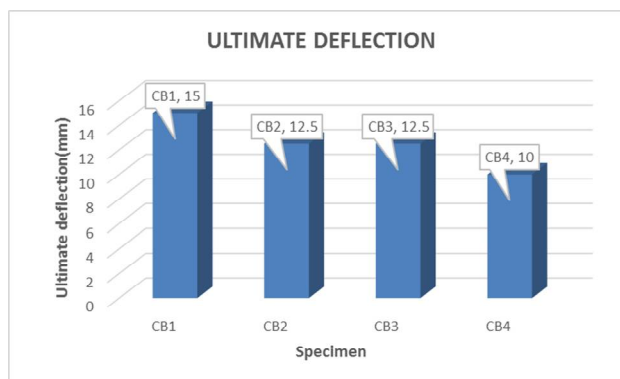


Fig 5: Comparison of ultimate displacement of tested specimen

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D. DUCTILITY RATIO

Ductility of a structure is its ability to undergo large inelastic deformations before failure. In limit and seismic design, the ductility of a member is usually expressed as the ultimate deformation to the deformation at first yield. Table III summarises the ductility ratios of the tested specimens. The specimens CB1 and has ductility ratio of 7.5 while the specimen CB2 with 2 bars bundled at the corners showed a ductility of and specimen CB3 with 3 bars bundled showed ductility of 4.16. The specimen CB4 has ductility less than other 3 specimens.

Table III: Ductility ratio of tested specimens

Specimens	Yield displacement (mm)	Ultimate displacement (mm)	Ductility ratio
CB1	2	15	7.5
CB2	2.5	12.5	5
CB3	3	12.5	4.16
CB4	4	10	2.5

E. STIFFNESS DEGRADATION

Stiffness is the load required to produce unit deflection of column. Stiffness of the column can be calculated as the ratio of load to deflection. The stiffness of the column specimen at the end of each cycle has been plotted to observe the stiffness degradation. The stiffness degradation plots are shown in the figure 6. Stiffness degradation was found to be less for the specimen CB1 in both cycles of loading. The variation for the specimen CB2 was almost similar to that of the control specimen CB1. But for the specimens CB3 and CB4 it was found to be higher than the other two specimens.

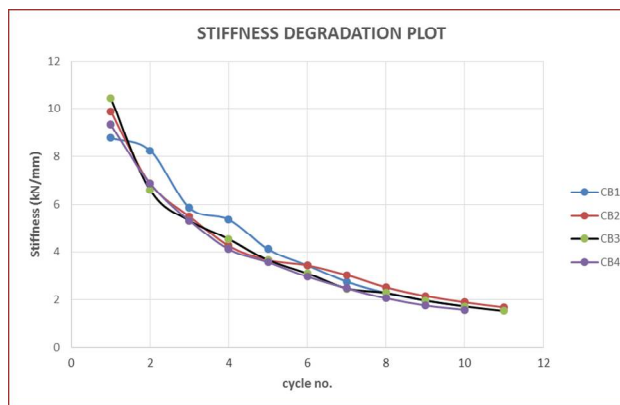


Fig.6. Stiffness degradation plot in the reverse loading cycle

F. CRACKING PATTERNS

The cracking patterns of the test specimens are shown in Fig.7. For the control specimen CB1 flexural crack was found to develop during fifth forward cycle on the left face and during fourth reverse cycle on the right face, and shear crack was observed during third reverse cycle on the front face while it was observed in the fourth reverse cycle in the rear face. For the specimen CB2, with two bars bundled, the flexural crack occurred during third reverse cycle on the left face, whereas it was during fourth reverse cycle on the right face. On the front face shear crack appeared in the fourth reverse cycle and in the fourth reverse cycle on the rear face. For the specimen CB3, with three bars bundled, flexural crack was seen during the fourth forward and reverse cycle for left and right faces respectively, whereas shear crack was seen during the fourth reverse cycle on both front and rear faces. For the specimen CB4, with four bars bundled,

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flexural crack was observed on the left face during fourth forward cycle of loading and on the right face in the third reverse loading cycle, while shear cracks developed in the third reverse cycle for both front and rear faces

Plastic hinge length are formed near sections experiencing maximum bending moments. For predicting the displacement capacity of a column it is necessary to estimate the plastic hinge length. Proper estimation of the length of the plastic hinge is very essential when a column is subjected to cyclic lateral load since plastic hinge region of columns is the region where large amount of inelastic energy is expected to dissipate by undergoing large inelastic deformations. Failure of a column which is subjected to cyclic lateral load is assumed to take place at the center of the assumed plastic hinge. Higher values for plastic hinge length indicates ductile failure and lower values indicates shear failure near the support. For the specimens CB3 and CB4 the plastic hinge length was very close to the joint between the column and stub. But it was shifted upwards in the case of CB1 and CB2, which indicated ductile failure. But for the specimens CB3 and CB4 the plastic hinge length was less, which means shear failure was predominant for these specimens and CB4. From the crack pattern it was observed that the damage zone in the control specimen was above 19 cm from the joint. While, in the case of the specimens CB2 and CB3 the damage zone shifted upwards by 3.5 cm and 2.5 cm, respectively. From the crack pattern it can be seen that there was no improvement in the flexural behaviour of the bundled specimens, while shear behaviour was found to increase for the specimens CB3 and CB4. Table IV shows the test results of post flexural shear failure of columns.



Fig. 7(a). Flexural cracks in CB1



Fig. 7(b). Shear cracks in CB1

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Fig. 7(c). Flexural cracks in CB2

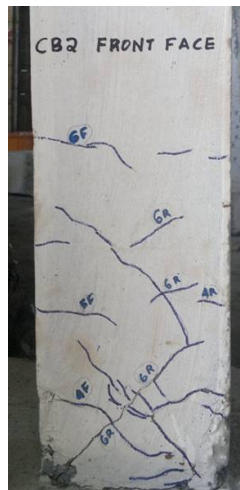


Fig. 7(d). Shear cracks in CB2

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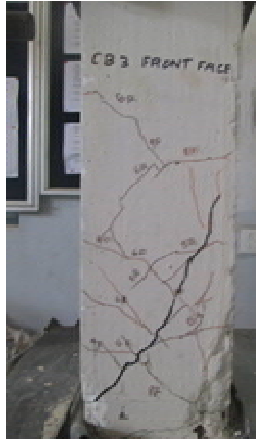


Fig. 7(e). Shear cracks in CB3

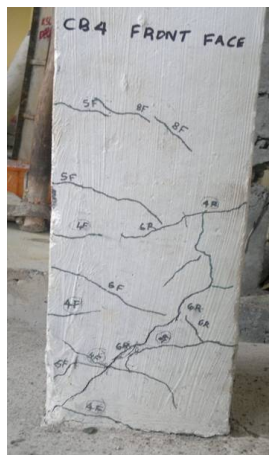


Fig. 7(f). Shear cracks in CB4

Table.IV: Test Results of post flexural shear failure of columns

Specimen	CB1	CB2	CB3	CB4
Ultimate Load (kN)	47.13	45.559	40.846	39.275
Length up to Centre of Plastic hinge from base (mm)	107	106	65	60
Length of Plastic Hinge (mm)	130	125	83	40

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IV. CONCLUSIONS

After testing the specimens under cyclic lateral loading and comparing the performance of the specimens, the following conclusions was drawn:

- The specimen with two bundled bars showed 11.64% increase and that with three bundled bars showed 9.74% increase in the energy dissipation capacity than the control specimen.
- The specimen with four bars bundled showed a decrease of 5.25% in energy dissipation capacity than the control specimen, which showed poor performance.
- Ultimate load carrying capacity and ultimate deflection of the control specimen and specimen with two bars bundled was higher than the other two specimen.
- The hysteretic response of specimens with conventional reinforcement arrangement and with two bars bundled was similar and showed better performance than the specimens with three and four bars bundled.
- Ductility was more for control specimen than the other three specimens, indicating flexural failure, which is desirable for columns
- Control specimen failed mainly by flexure than shear, whereas shear failure was predominant in specimens with two and three bars bundled.
- The specimen with two bar bundled had similar failure pattern as that of control specimen.
- Shear failure in specimens with three and four bars bundled were due to lack of bars in between bundled bars.
- Lateral load carrying capacity of specimen with three and four bars bundled was less due to lack of confinement pressure in the concrete core.
- It could be concluded that column with conventional reinforcement arrangement showed better performance in terms of ultimate deflection, ultimate lateral load carrying capacity, energy dissipation capacity and cracking pattern. Also the specimen with two bars bundled was comparatively better.
- Columns with more than two bars bundled affects the cyclic performance of columns

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