EXPERIMENTAL SEISMIC ASSESSMENT OF FULL SCALE NON-SEISMICALLY DETAILED RC STRUCTURE BY PUSHOVER METHOD

Akanshu Sharma[1], G.R. Reddy[1], R. R. Babu[2], D. Revanna[2], K.K. Vaze[1]
[1]*Reactor Safety Division, Bhabha Atomic Research Centre, Mumbai 400085, India*
[2] *Central Power Research Institute, Bangalore 560080, India*
E-mail of corresponding author: akanshu@barc.gov.in

ABSTRACT

As more and more emphasis is laid on nonlinear analysis of RC framed structures subjected to earthquake excitation, the research and development on nonlinear static (pushover) analysis as well as nonlinear dynamic (time history) analysis is in the forefront. For validation of the analytical procedures to develop the lateral load-displacement (pushover) curve, in this work, an experiment was performed on a 3-D full-scale structure four storey structure. The structure, having one bay along both horizontal directions, was loaded under monotonically increasing lateral pushover loads. The structure tested was the replica of a part of the existing office building in Mumbai. The part of the structure was deliberately selected to have certain eccentricities and the reinforcement details were kept as per non-seismic standards in India. The details of the structure, loading pattern, test setup along with the experimental results in the form of base shear v/s roof displacement curves at various floors, deflected shape of the structure at different levels of base shear, failure modes, strains, rotations etc. are reported in this paper. The failure patterns clearly displayed the vulnerability of RC buildings with non-conforming detailing to fail in undesirable failure mechanisms such as joint shear failures.

INTRODUCTION

Due to prohibitive computational time and efforts required to perform nonlinear dynamic analysis, researchers and designers all over the world are showing keen interest in nonlinear static pushover analysis. Codes such as ATC 40 [1], FEMA 273 [2] followed by FEMA 356 [3] and more recently FEMA 440 [4] have given detailed guidelines to perform the nonlinear static analysis and to use it to obtain the performance of structures under given earthquake scenario. The post processing procedures recommended to determine the performance of the structure against a given earthquake are different for different codes but all these procedures require determination of nonlinear force-deformation curves that are generated from pushover analysis. This simplifies the structural model while providing insight information about the likely nonlinear behavior of the structure. Therefore, a vital step towards good seismic performance estimation of the structure is reliable and accurate determination of force-deformation curve popularly known as “pushover curve” or “capacity curve”.

The validation of the analytical procedure requires comparing analytical results with those of the experiments. The experiments on full-scale real life type structure is the best way to not only study the behavior of the structures under lateral seismic loading but also these results can provide excellent database to validate the analytical procedures. Efforts have been made in past to perform tests on full-scale structures [5,6,7,8] but the database is not too large due to prohibitive cost, time and efforts involved. In this work, a full-scale four storey reinforced concrete structure was tested under monotonically increasing lateral pushover loads with an inverse triangular loading pattern. The structure tested was the replica of a part of the existing office building in Mumbai. The part of the structure was deliberately selected to have certain eccentricities and the reinforcement details were kept as per non-seismic standards in India.

The results of the test show that the structures designed and detailed as per non-seismic practice tend to undergo undesirable failure modes such as joint shear, torsion and bond slip.

DESCRIPTION OF STRUCTURE

Geometry

In order to keep the structure as close to reality as possible, no special design for the structure as such was performed and instead a portion of a real life existing office building was selected and replicated. Thus the structure tested in this work was a replica of a part of an existing office building. The portion was deliberately selected so that it had certain eccentricities and was un-symmetric in plan (Fig.1). Also the column sizes and sections were varied along the storey as in the case of original real life structure. The structure tested was a four storey structure with a typical column height of 4 meters. The plan dimensions were 5m x 5m. Fig.1 shows the general geometric arrangement of the structure. The typical beam size was 230mm x 1000mm and the column size varied from 400mm x 900mm to 300mm x 700mm as shown in Table 1. The slab thickness was 130mm. In Table 1, the longitudinal reinforcement for the beams is mentioned in the “number of bars – diameter of bars in mm (location of reinforcement in the section)”, e.g. 2-16 (Top) refers to 2 number of 16mm diameter bars located at the top of the section (to act as compression reinforcement under sagging moment). The longitudinal reinforcement for the columns is distributed uniformly along the periphery and is mentioned in the “number of bars – diameter of bars in mm” format e.g. 12-28 refers to 12 numbers of 28mm diameter bars distributed uniformly along the periphery of the column. The transverse reinforcement is mentioned in “diameter of stirrups/ties (in mm) – spacing of stirrups/ties (in mm), e.g. 8-200 refers to 8mm diameter bars as stirrups/ties spaced at a centre to centre spacing of 200mm.

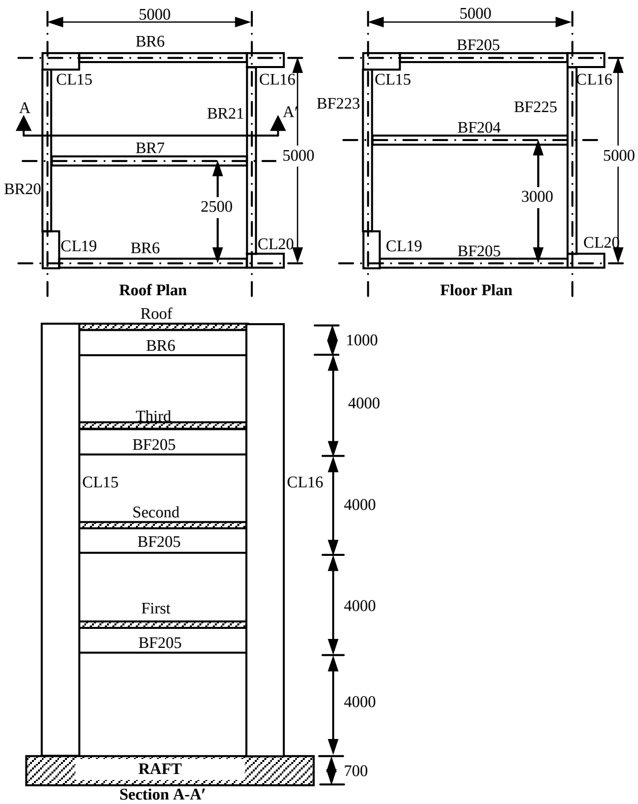


Fig.1 Geometry of the Structure

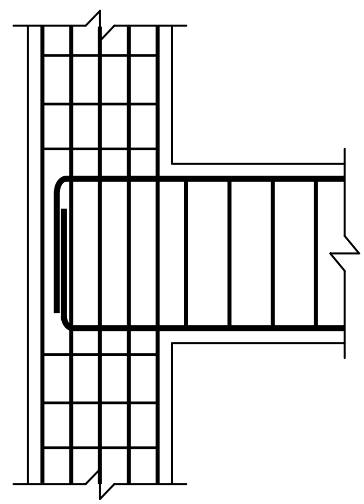


Fig.2 Typical joint details provided in the structure

Table 1 Details of Structural Members

Beam/Column	B (mm)	D (mm)	Long. Reinforcement	Trans. Reinforcement
BF 204	230	1000	2-16 (Top); 3-16 (Bottom)	8-200 c/c
BF 205	230	1000	2-25 (Top); 2-25 (Bottom)	10-125 c/c
BF 223	230	1000	2-25 (Top); 2-25 + 1-16 (Bottom)	10-125 c/c
BF 225	230	1000	2-20 (Top); 2-25 (Bottom)	10-150 c/c
BR 6	230	1000	2-20 (Top); 3-20 (Bottom)	8-200 c/c
BR 7	230	600	2-16 (Top); 3-16 (Bottom)	8-120 c/c
BR 20	230	1000	2-20 (Top); 2-25 (Bottom)	8-175 c/c
BR 21	230	1000	2-20 (Top); 2-20 (Bottom)	8-175 c/c
CL15/ CL19 (Grd to 2nd)	400	900	12-28	10-100 c/c
CL15/ CL19 (2nd to 3rd)	400	700	4-25 + 6-20	10-100 c/c
CL15/ CL19 (3rd to 4th)	300	700	8-20	10-100 c/c
CL16/ CL20 (Grd to 2nd)	350	900	12-25	10-100 c/c
CL16/ CL20 (2nd to 4th)	350	900	10-20	10-100 c/c

Material Properties

For each floor level and for columns extending from one floor to another, six standard 150mm cubes were tested under compressive loads and the average 28 day strength was obtained. The average concrete strength was obtained as 30.86 N/mm^2. Cold worked deformed bars with a nominal strength of 415 MPa as per IS 1786:1985 [9] were used in construction. The average yield strength for the bars was obtained as 502.20 N/mm^2 and the average ultimate strength was obtained as 615.30 N/mm^2.

DESIGN AND CONSTRUCTION OF STRUCTURE

General description

Though, the original structure was detailed as per the confined (conforming) detailing practice as per IS 13920:1993 [10], whereas unconfined (non-conforming) detailing was adopted for the structure tested under the program. This was done, keeping in the mind the fact that mostly, the structures for which Pushover analysis is performed are old structures needing seismic re-qualification and retrofitting, which generally will be following non-conforming detailing. Moreover, it presents a more severe condition from analysis point of view. Also, since the structure tested is replica of a small portion of the larger original structure, the continuous reinforcements in the slab and beams were suitably modified to fit as per the requirement. Fig.2 shows a typical non-conforming joint detail as was provided in the structure. The beam longitudinal reinforcement bars were extended beyond the face of the column into the joint up to a length equal to the development length for the bar as calculated by Indian standard code of practice, IS 456:2000 [11].

Foundation

One of the major challenges in the task was to restrict the possible rotation of the foundation of the structure. This was practically not possible, if the isolated footings were provided. Therefore, foundation for the structure was provided as a common raft for all the four columns. The substratum was found to be hard rock and therefore, in order to avoid any possible rotation of the foundation, rock anchors were provided. In total, 144 numbers of 1.5 m long rock anchors were provided with 700mm embedment in concrete and 800 mm in rock. The raft was proportioned in such a way that the clear overhang of the raft is equal to 750mm from the face of each column on both sides. Thus, the raft size was 7.40 m x 6.73 m. Fig.3 shows construction of foundation. The superstructure was cast in stages as any other normal building construction with a quality control at par with the general quality control followed during the construction of normal residential buildings in India.

Fig.3 Construction of Raft Foundation with Rock Anchors

Loading Arrangement

The load on the structure was applied using tower test facility available at Central Power Research Institute (CPRI) Bangalore. The load application pattern was kept as inverted triangular with a load of P:2P:3P:4P corresponding to 1st floor:2nd floor:3rd floor:4th floor. The basic mode of application of load was by pulling the structure. The loading arrangement using steel plates was provided in the slab. The load was applied remotely by means of high strength cables passing through pulleys using electro-mechanical winches controlled through programmable logic control PLC (SCADA) system. The loads applied were continuously monitored using tension-type load cells.

EXPERIMENTAL SETUP

Test Facility

The test was conducted at tower testing facility of CPRI, Bangalore. The facility is generally and regularly used to perform monotonic load tests on full scale transmission line towers. The test facility is well equipped with high strength cables, pulleys, calibrated load cells, electro mechanical winches with PLC control for accurate and simultaneous load application in pre-defined pattern. However, the facility could perform the test only in load-control mode. This may not truly be a limitation since the pre-peak curve is generally agreed to be more accurate in case of load-control, though a displacement control is required to capture post-peak degradation. Therefore, it would be best to perform the test under load-control in pre-peak region and under displacement control in post-peak region. However, keeping in mind the technical capabilities of the facility and also financial and time considerations, the whole experiment was conducted in load-control mode. Fig.4 shows structure being tested at the tower test facility.

Fig.4 (a) Structure at Tower Testing Facility (b) Structure during test

Instrumentation

The instrumentation used to obtain the required information about the behavior of the structure included

(i) Load Cells to monitor and apply the load on the structure in controlled manner.
(ii) Digital theodolites on either side of the structure (one toward CL 16 and one towards CL 20 side), to measure displacements and laser based displacement measuring devices to record corresponding displacement.
(iii) Strain gauges on reinforcement bars to obtain strain data.
(iv) Tilt meters for measuring member and joint rotations. These were mounted directly on the structure at the beam and column intersecting at the joint.
(v) Digital dial gauges to provide information on surface strains at the base of the columns at raft level

Loading Sequence

The loading sequence during the test was kept such that the load in the first floor was increased in the steps of 1t (9.81 kN). Thus, the load in the second floor was incremented with the steps of 2t (19.62 kN), that in 3^{rd} floor in steps of 3t (29.43 kN) and in 4^{th} floor in steps of 4t (39.24 kN). Thus the ratio of 1:2:3:4 is always maintained.

EXPERIMENTAL RESULTS

The pushover curves as obtained for CL16 side and CL20 side are shown in Figs 5 and 6 respectively. Since the experiment was conducted under load control, the drooping part of the curves could not be obtained. As can be seen from the two figures, the maximum displacement for CL16 side was obtained as 537mm and that on CL20 was obtained as 765mm. This clearly demonstrates torsion due to eccentricity raised from column orientation. The difference in the curves is as expected showing more displacement on CL20 side due to less stiffness offered by CL19 in loading direction. The average top drift is therefore equal to around 4% of the total height of the building.

The structure behaved linearly till a base shear value of around 300 kN. At this point the flexural tension cracks at the base of the columns started to get generated and the structure displayed a reduced stiffness. After reaching a base shear value of approx 500 kN, the cracks at the base of the columns opened wider and failures at other locations namely beams and beam-column joints started to show up. As a result the stiffness of the structure further went down, as can be seen from the pushover curves. After reaching the base shear values of 700 kN, the joints of the structure displayed rapid degradation and the inter-storey drift increased rapidly. On further increase in the lateral load, the structure displayed a very soft behavior with large displacement increase for the same increase in the base shear. After reaching a base shear of 90t (882.90 kN), i.e. 9t load at first floor, 18t at second floor, 27t at third floor and 36t at fourth floor, the structure started undergoing increasing displacement and the resistance offered by the structure started to reduce.

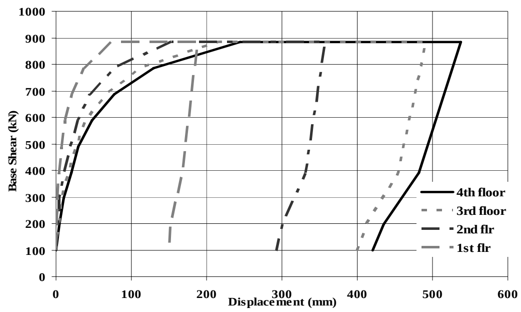


Fig.5 Pushover Curve for CL 16 Side

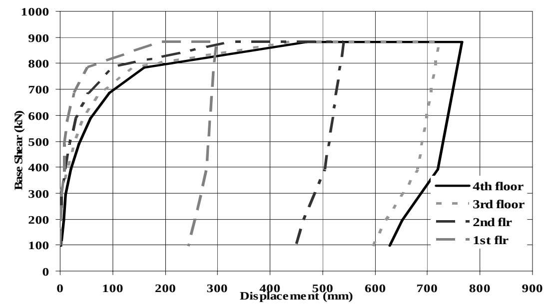
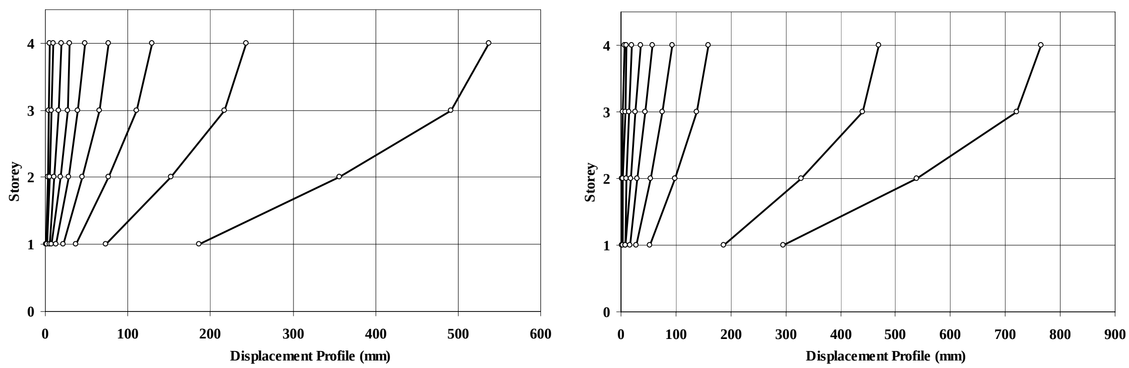


Fig.6 Pushover Curve for CL 20 Side



(a) CL 16 side (b) CL 20 side

Fig.7 Displacement Pattern for Increasing Top Drift for the structure

Fig.7 shows complete displacement profile of each storey with respect to top displacement level. Each curve corresponds to the displacement profile of a load step. The first load step is depicted by the very first curve on extreme left with the curves corresponding to further load steps to the right of the curve of previous load step with the last step (ninth step) corresponding to a base shear of approx 900 kN is shown in extreme right. Initially, when the structure was loaded, it went fairly linearly till the third load step corresponding to a base shear of 300 kN. As the lateral load on the structure was increased, the inter-storey drift increased and the structure went into inelastic (nonlinear) range. It was observed that as the displacement increases, the contribution of relative displacement between third and fourth floor reduces, which is attributed to the joint failure at the third floor level.

FAILURE PATTERNS

Fig.8 shows the failure of bottom storey columns on (a) compression side and (b) Tension side. The failure patterns are typical for the reinforced concrete members subjected to combined axial load and uniaxial bending. Compressive loads along with high bending moments result in, crushing of concrete on front face of the column and tension cracks on rear face (Fig.8 (a)). Tension loads along with bending moment result in cracks from the rear face of the columns that grew, as the load increased, towards the front face of the columns. The spalling on the front face was nominal compared to that of compression side columns (Fig.8 (b)).

Fig.9 (a) shows the failure mode of the beam BF 205 connected to CL 15 at 1st floor in flexural mode combined with bond slippage of the beam tension reinforcing bars. Due to lateral loading, the bending moments were generated in the beam with hogging moments towards the end fixed with column CL16 and sagging moments towards the end fixed with column CL15. As a result, flexural tension cracks could be seen initiating from the soffit of the beam and propagating towards the slab. Due to high tensile stresses generated in the beam bottom bars, a slippage of the bars seems to have occurred. Spalling of concrete was observed on both the tension and compression face of the beam due to extensive cracking and crushing respectively. Fig.9 (b) shows the failure of the beam BF 223 transverse to the direction of loading. As the lateral load increased, the beams transverse to the direction of loading (BF223) were pushed by the slab. This push was resisted by the stiffness provided at the ends due to restraining action of columns CL16 and CL 20. Due to the end restraints, the beams suffered high compatibility torsion moments at the fixed ends.

(a) Compression side column, CL 16

(b) Tension side column, CL 15

Fig.8 Failure mode of columns at ground floor

(a) Beam BF 205 at 1st floor, flexure failure

(b) Beam BF 223 at 1st floor, torsion failure

Fig.9 Failure of Beams under different failure modes

Figs.10 through 13 show different types of joint failures observed in the structure. Under the action of lateral forces, beam-column joints are subjected to large shear stresses in the core. Typically, high bond stress requirements are also imposed on reinforcement bars entering into the joint. The axial and joint shear stresses result in principal tension and compression that leads to diagonal cracking and/or crushing of concrete in the joint core. The flexural forces from the beams and columns cause tension or compression forces in the longitudinal reinforcements passing through the joint. During plastic hinge formation, relatively large tensile forces are transferred through bond. When the longitudinal bars at the joint face are stressed beyond yield, splitting cracks are initiated along the bar at the joint face. If the cover to the reinforcement bars is less and if the joint core is not confined by confining reinforcement in the form of stirrups, the cover concrete is spalled off due to the pressure exerted by the beam reinforcement bars. Most severe joint failures were found in the case of column CL 19. This might be attributed to the relatively low column depth (400mm) than beam depth (1000mm). In such cases, plasticization of columns can occur which may also lead to damage ingress in the joint core. Moreover, there was high eccentricity between beam and the column since the beam of width 230mm was flushed with the face of the column with the width of 900 mm.

Fig.10 shows the failure of joint of CL 19 at first floor. High stresses in the joint resulted in diagonal cracks in the core followed by cover spalling due to the pressure exerted by the beam longitudinal reinforcement. Fig.11 shows the failure of joint of CL 19 at 2nd floor level, which shows the beam bar bursting out of the joint. This is a typical failure mode for joints with unrestrained bars. This occurred since in order to provide the development length of the beam main reinforcement, the bent bars had a long free length beyond the bent and there were no transverse reinforcement to provide any restrain to the same. Such a failure can, in general, be prevented if proper confining reinforcement is provided in the joint core. Fig.12 shows the failure of the joint of CL16 at first floor level that exhibited bond failure along with beam flexural failure and spalling of side cover due to pressure exerted by the reinforcement. High tension force in the beam reinforcement resulted in bond deterioration and ultimately failure with splitting of concrete. Also, large cracks along with spalling of concrete can be seen at the beam-column interface. Fig.13 shows diagonal shear crack in the joint of CL20, 2nd floor during the test with flexural cracks in the beam and bond failure of the tension reinforcement. It can be observed that a clear diagonal shear crack appeared in the joint during the test but it was not further opened and the failure essentially got transferred through bond mechanism. Although, the beam longitudinal reinforcement was bent up to the required development length inside the column, it indicates that such development by bending in the re-bars may not be good enough to prevent the bond failure.

Fig.10 Joint failure of CL 19 at 1st floor

Fig.11 Joint failure of CL 19 at 2nd floor

Fig.12 Joint failure CL16, 1st Floor

Fig.13 Joint Failure CL20, 2nd floor

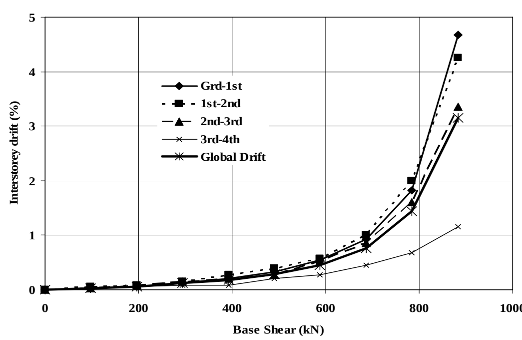


Fig.14 Inter-storey drift as a function of base shear

Inter-storey Drift

Fig.14 shows the inter-storey drift between ground to 1st floor, 1st to 2nd floor and so on as a function of base shear on CL 16 side. Also, in the same plot, the global drift obtained as the lateral roof deflection divided by the total height of the structure expressed as percentage is given. As seen by the graph, maximum inter-storey drift were obtained between the ground to first floor and first to second floor and were of the order of 4.5%. The same between second to third floor was around 3.5%, which was also the order of global drift. The inter-storey drift between the third and roof level were around 1-1.5%, which shows that most of damage was concentrated within lower floors.

CONCLUSIONS

Experiment was conducted on a full-scale RC framed structure which was a replica of a substructure of an existing office building. The structure was constructed with non-seismic detailing so that it can be closer to the most of existing, old RC structures worldwide that may need seismic re-qualification. The foundation was constructed with rock anchors to avoid the possibility of its rotation at the time of experiment. The failure patterns clearly displayed the vulnerability of RC buildings with non-conforming detailing to fail in undesirable failure mechanisms such as joint shear failures, bond failures etc. Although the structure displayed large variety of failure mechanisms, the damage mostly was concentrated in the joint region or at the beam/column interfaces with wide flexural cracks along with bond failures. The severe damage in the joints at first floor level as well as, but more moderate, at second floor level combined with the hinging of column base sections at the ground level, would apparently lead to a soft storey-type mechanism. However, it is evident from the experimental deformed profile shape that a pure soft storey mechanism did not occur at the first floor. The observed global mechanism, related to joint damage can be explained with the concept of "shear hinge" [12] that states that for a given inter-storey drift demand, the occurrence of a shear hinge, through shear cracking of the joint region, might lead to a concentration of deformation demand in the panel zone, with significant reduction of rotation demand in the adjacent beam or column critical sections. The inter-storey drift demand is thus spread between the storey above and the storey below the joint, somewhat delaying the occurrence of single soft-storey mechanisms.

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