Behaviour of seismic and non-seismic RC frames under cyclic loads

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and U. Ramesh

Two one-third scale models of reinforced concrete portal frames were tested under slow cyclic loads to evaluate the relative hysteretic performance of conventional non-seismic detailing as per IS 456 : 2000 and seismic ductility provisions as per IS 13920 : 1993. Though both frames exhibited stable hysteretic behaviour, the inelastic activities were concentrated in the beam-column joints rather than at beam ends. Premature cracking and spalling of concrete on the outside face of the column due to loss of anchorage of beam bars made them ineffective in transferring the tensile forces and the full strength of the frame members could not be achieved. Also, due to the small volume of the joint panel, the compressive stresses in the joint concrete were excessive enough to cause severe cracking of the joint concrete. These tests clearly indicate the inadequacy of seismic detailing of the current IS 13920 : 1993 which does not have specific provisions for the design and detailing of beam-column joints.

Keywords: *Slow-cyclic testing, RC moment frames, seismic detailing, knee joints, joint shear stress, anchorage.*

Earthquake resistance of reinforced concrete (RC) moment resisting frames (MRF) comes from their ability to deform well into inelastic range and dissipate the seismic energy through the stable hysteretic behaviour. These inelastic deformations are mainly concentrated in certain critical regions of the structure, that is, in beams near the beam-column joints which are suitably designed and detailed to undergo large inelastic deformations. Furthermore, as per global practice, the resulting yield mechanism for entire structural frame should be such that it can sustain large drifts without collapse, by distributing the inelastic demand uniformly across its various members. Therefore, ductile MRFs are designed in such a way that the plastic hinges develop at the ends of beams, and columns remain elastic except at the base of frames, that is, strong-column-weak-beam yield mechanism. In contrast, plastic hinge formation in columns prior to beams causes a sidesway collapse mechanism which leads to a rather abrupt collapse of the structure (that is, an unsatisfactory low ductile response). In India, a large number of RC structures are single-storeyed where sidesway collapse mechanism can be accepted if the column axial loads are low and failure of beam-column joints is prevented.

In addition, the beam-column joint regions need to be suitably detailed so that inelastic activities are not only kept out of the joint but also joint deformations are minimum. External beam-column joints and knee (or top-of-column) joints pose further complications over interior ones. In single-storeyed structures, the performance of knee-joints becomes crucial for the overall behaviour of the frame under lateral loads. Unfortunately, the state-of-knowledge about the design and behaviour of these knee-joints is far behind in comparison to interior joints[1,2]. Further, the question of spalling in the joint region has been a subject of extensive research. Frame joints with only longitudinal reinforcement are known to undergo diagonal tension and hence sustain pull out as a triangular piece under closing moments, even when adequate development of the longitudinal bars is provided[3,4]. Furthermore, most buildings in highly seismic regions in India are constructed in accordance with the general RC construction code IS 456 : 2000 as the usage of seismic codes is extremely limited[5]. The performance of such 'non-seismic' frames is questionable in future earthquakes. Therefore, it was considered important to evaluate the performance of conventionally designed non-seismic moment frames in terms of its strength, ductility and energy dissipation capacity under lateral loads.

An experimental investigation was carried out to study the hysteretic behaviour of two portal frames under slow cyclic loads, with different detailing provisions, one with conventional non-seismic detailing as per IS 456 : 2000 and IS SP 34 (S&T) : 1987, and another with ductile detailing provisions required for earthquake resistant design, as specified in IS 13920 : 1993[5,6,7]. The objective of this paper is to report the findings of experiments with regard to their relative performance measured in terms of available ductility, lateral load-resisting capacity and to compare the inelastic activities and failure mechanisms.

Description of test specimens

A typical single-storey RC structure considered for the present study was assumed to be located in the highest seismic zone (Zone V) of IS 1893 : 2002[8]. The details of the study building are described by Ramesh[9]. Since testing of the complete structure was not possible in the laboratory, only a single bay, two frames were modelled at a reduced scale of 1:3 as shown in *Fig* 1. This one bay portal frame, consisting of two frames, possessed sufficient lateral stability due to a slab which also helped transfer the forces to the frames. The slab had a square opening (430 mm x 430 mm) at each of the corners near the beam-column junctions to avoid possible interference of the slab in the formation of beam plastic hinges. *Fig* 2 shows the proportions and reinforcement detailing of the models used in the testing programme. High yield strength deformed (HYSD) bars (Fe 415) were provided for longitudinal reinforcement and mild steel bars (Fe 250) were for lateral ties (stirrups), footing and slab reinforcement as HYSD reinforcing bars were not available in required diameters.

Materials

The prototype materials were used to design and fabricate model specimens. The concrete used was of M20 grade. HYSD steel bars (Fe 415) were used for longitudinal reinforcement and mild steel bars (Fe 250) were used for lateral ties (stirrups), footing and slab reinforcement. For the Fe 415 reinforcement the average yield value was 455.4 MPa and the ultimate value was 565.3 MPa. The concrete was made from ordinary Portland cement (Grade 43), Solani river sand, and crushed stone of maximum size not exceeding 8 mm. These materials were mixed in proportion of 1:1.5:2.75 by weight, respectively and the water-cement ratio was kept at 0.5. The 28-day average compressive strength from 100 mm cube tests were: 33.2 MPa (non-seismic model) and 28.9 MPa (seismic model).

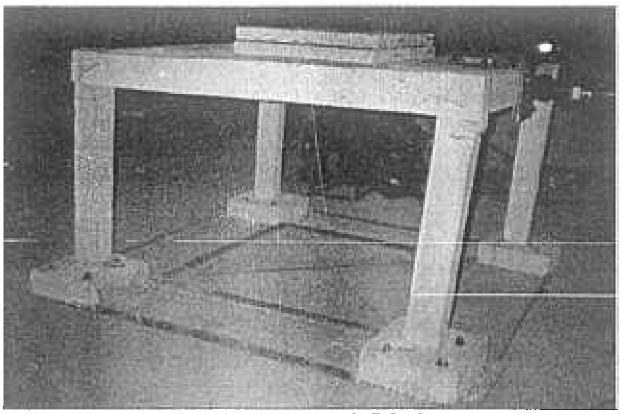

Fig 1 One-third scaled model of RC frame used in the experimental study (seismic model at a drift ratio of 6 percent)

Reinforcement details

Beams and columns

Reinforcement details of the two specimens primarily differed in two areas: *(i)* anchorage of longitudinal bars and *(ii)* detailing and spacing of transverse steel. Ductile detailing of RC structures for seismic forces as per IS 13920 requires longer anchorage length for longitudinal bars. For example, the top and bottom beam bars shall be provided with anchorage length equal to development length in tension plus 10 times the bar diameter minus allowances for bends and it will be measured from the inner face of the column. IS:13920 specifies that special confining steel in beam is provided over a length of twice the effective depth, *d*, of the beam at either end and the spacing of lateral ties shall not exceed (i) *d/4*, (ii) 8 times the diameter of the smallest beam bar and (iii) not less than 100 mm. Elsewhere in beam, the spacing of ties does not exceed *d/2*.

The special confining reinforcement in column was provided on the either side of joint face over a length which was not less than *(i)* larger lateral dimension of the column section, *(ii)* one-sixth of clear span of the column , and *(iii)* 450 mm. The spacing of ties did not exceed one-fourth of the least lateral dimension of column section but was not less than 75 mm nor greater than 100 mm. Elsewhere in the column, the spacing of hoops was kept less than half of the least lateral dimension of the column section. Further, the area of cross section of bar used for lateral ties for special confinement, A_{sh}, shall not be less than

$$A_{sh} = 0.18Sh\frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right] \quad (1)$$

where,

S = spacing of hoops
f_{ck} = characteristic strength of concrete
f_y = yield strength of hoop steel
A_g = gross area of column cross section
A_k = area of confined core in the rectangular
h = longer dimension of rectangular hoop.

The rectangular hoop was a closed stirrup with a 135° hook which extended at least 10 times the bar diameter but not less than 75 mm at each end into the confined core.

Beam-column joints

IS:13920 neither limits the joint shear stress nor requires that joint steel to be computed as per shear demand in the joints. For 'unconfined' joints, the special confining reinforcement at the column ends continue through the joint core, whereas only half of it will be required for the 'confined' joints. The

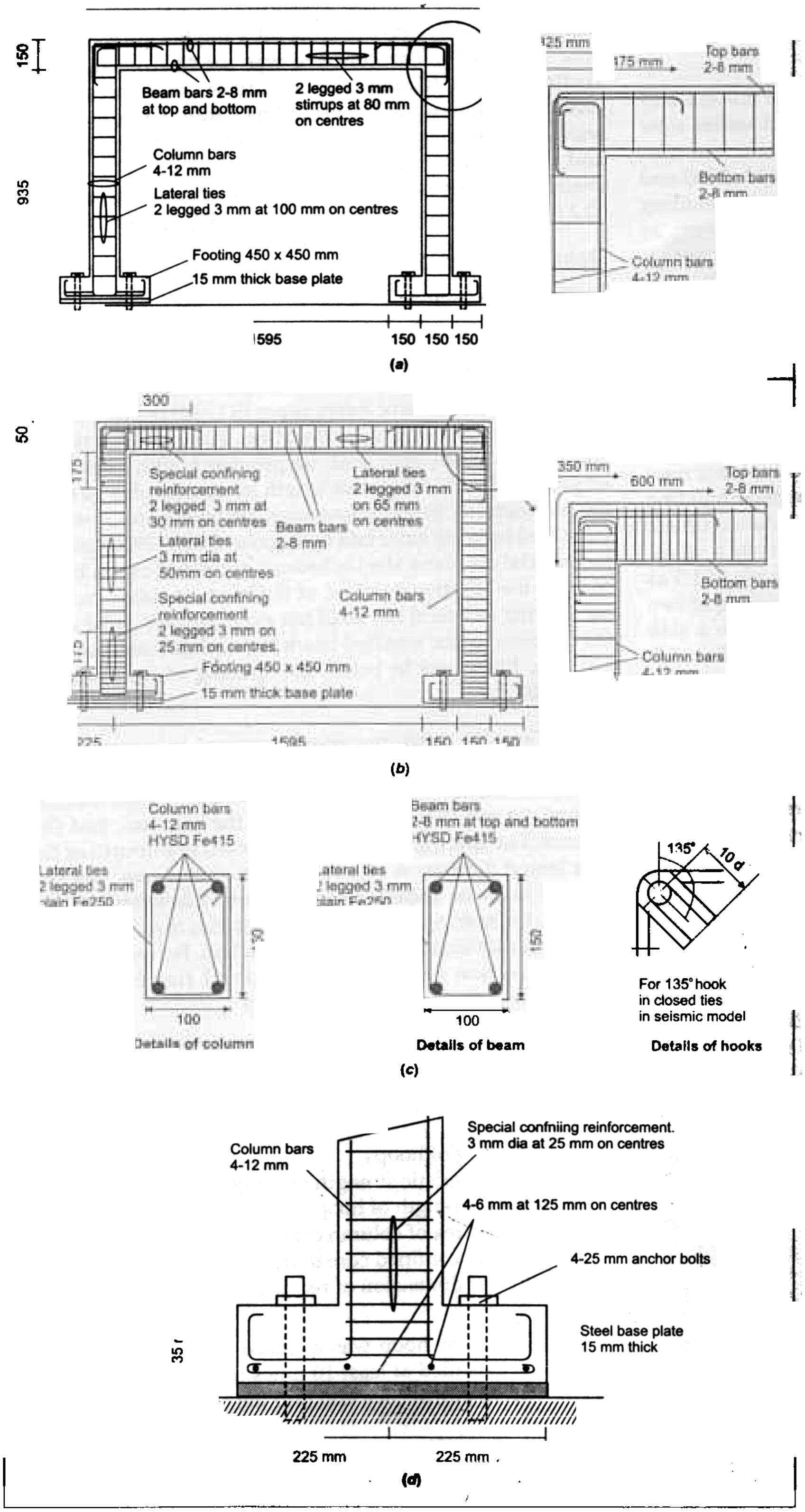

Fig 2 Reinforment details of RC frames ***(a)*** **non-seismic detailing in accordance with IS 456,** ***(b)*** **seismic detailing in accordance with IS 13920** ***(c)*** **anchorage details of** column, beams and hooks and (d) seismic details of footings

joint is considered 'confined' when it has beams framing into it at all vertical faces and beam widths are not less than three-fourth of the column width. *Fig* 2 shows reinforcement details of the joints for both specimens.

Loading history

For the general performance evaluation, loading history for the slow cyclic load tests was such that it could capture the critical issues of the structure's capacity as well as seismic demands. In the inelastic regime, capacity varies with the number of inelastic excursions/cycles and the magnitude of each cycle. Due to randomness of the seismic demands and the dependence of capacities on the demands, a single test or even a series of tests may not provide all the information needed for a seismic performance assessment. Therefore, a simple multiple step loading history was used which maximised information with minimum loading complexities[10].

The loading history for the test frames was pre-determined which consisted of a series of step-wise increasing load/deformation cycles. Loading cycles were kept symmetric, as the strength of the test specimen was the same in both directions, except for the last two cycles when the available stroke of the actuator in one direction was limited. In the beginning, loading was applied in load controlled mode which was switched to displacement-controlled mode after the occurrence of the first major inelastic activity. In order to obtain stable and reliable values of stiffness properties, a number of cycles in the elastic range were considered. Two cycles in each step of load/deformation were performed. Typical displacement loading consisted of two symmetric cycles with peak excursions being 2, 4, 6, 8, 10, 20, 40, and 50 mm in both directions followed by asymmetric cycles of 50 mm in one direction and 60, 80, 100 and 120 mm in the other direction. The load was applied by a 100-kN, 125-mm stroke length servo-hydraulic actuator at the slab level at the centre of beam span. The displacement of the frame was measured by two linear variable differential transformers (LVDTs) at the top of each frame as well as by the LVDT in the actuator's arm.

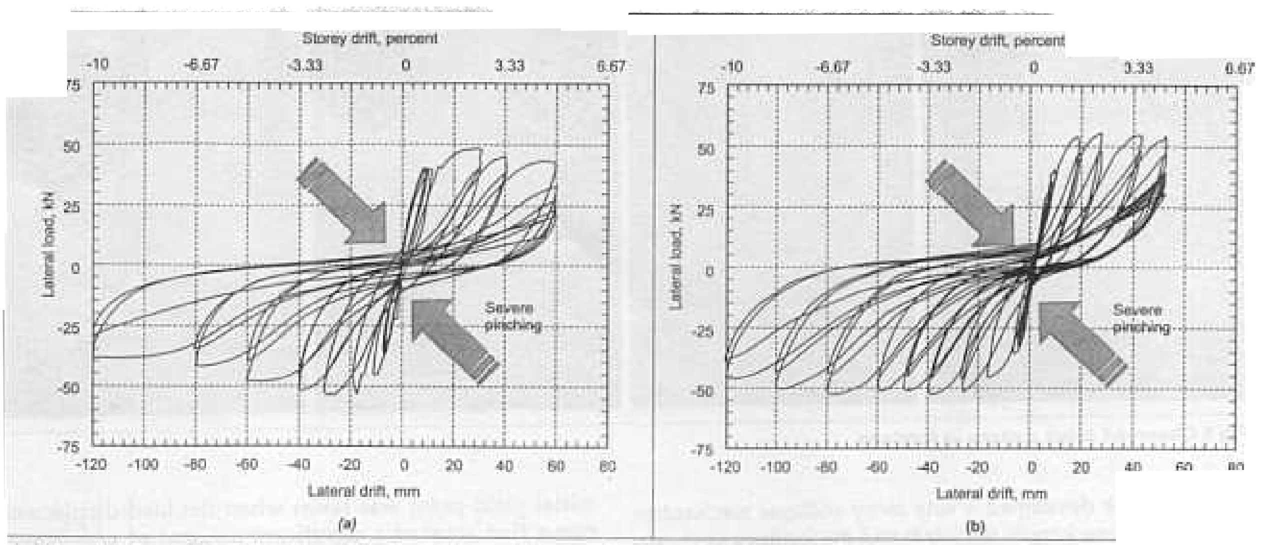

Fig 3 Load deformation behaviour of test specimens (i) non-seismic model (IS 456) and (ii) seismic model (IS 13920)

Behaviour of frames under slow cyclic loads

Load-deformation response

A stable hysteretic behaviour of the RC frame was observed in every stage of loading until displacements became as large as ±60 mm (5 percent drift ratio), *Fig* 3. In the early stages, the load-deformation curves were linear and diverged from the linearity following cracks in footings. Non-seismic and seismic model attained maximum loads of 61.2 kN and 53.6 kN respectively, which gradually reduced in subsequent cycles of loading of increasing drifts.

In the non-seismic model, the first cracking was observed in the footing in the 5th cycle of loading, which caused a sudden jump in the displacement from 3.0 mm to 3.98 mm at the constant load. In the case of seismic model, the first crack also appeared in the footing when the displacement in the positive excursion reached 4.23 mm at the load of 28.6 kN.

These cracks were formed above those longitudinal reinforcing bar of columns and extended into the footing. These cracks were formed due to the tensile stresses at the top face of the footing where no reinforcement was provided and it caused substantial reduction in the lateral stiffness of the system. The seismic code IS 13920 : 1993 does not explicitly specify flexural steel at the top face of footing where tensile force can develop under the action of lateral reversed cyclic loads. It is probably true that the cracks developed at top of footings because of rotational restraint offered by the anchorage bolts and simulated a boundary condition close to 'fixed' rather than 'pinned'. In reality, however, one may expect much lower rotational restraint offered by the surrounding soil to the footing and such cracking can not be completely ruled out even with lower rotational restraints.

With the increase in drift loading, there were further cracking of concrete in the joint regions and in the footings. Severe pinching of hysteretic loops were observed at large displacement cycles, primarily due to slippage of reinforcing bars in the beam-column joint region following cracking and spalling of concrete on the face of the column. As seen in *Fig* 4, considerable spalling of concrete was observed in the non-seismic model.

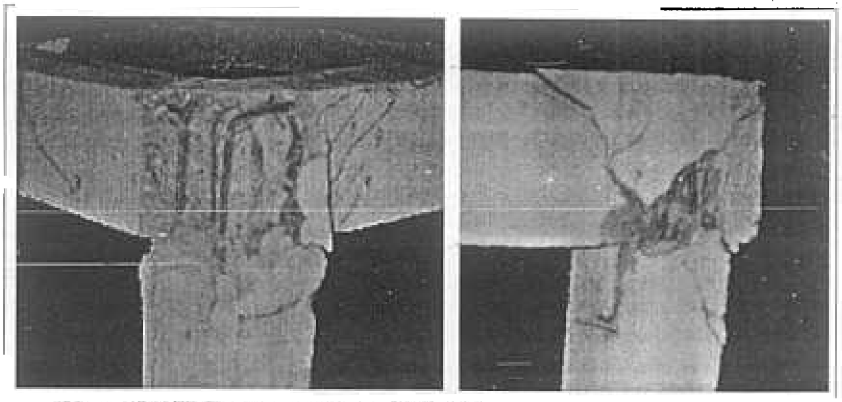

Fig 4 Spalling of concrete and damage to beam-column joints of the non-seismic frame

Damage pattern and failure mechanism

The inelastic activities in the frame were concentrated in the beam-column joints and footings rather than at the beam ends and near the column bases. This concentration of damage at the beam-column joints and the footings was highly undesirable and detrimental to overall lateral response of the structure.

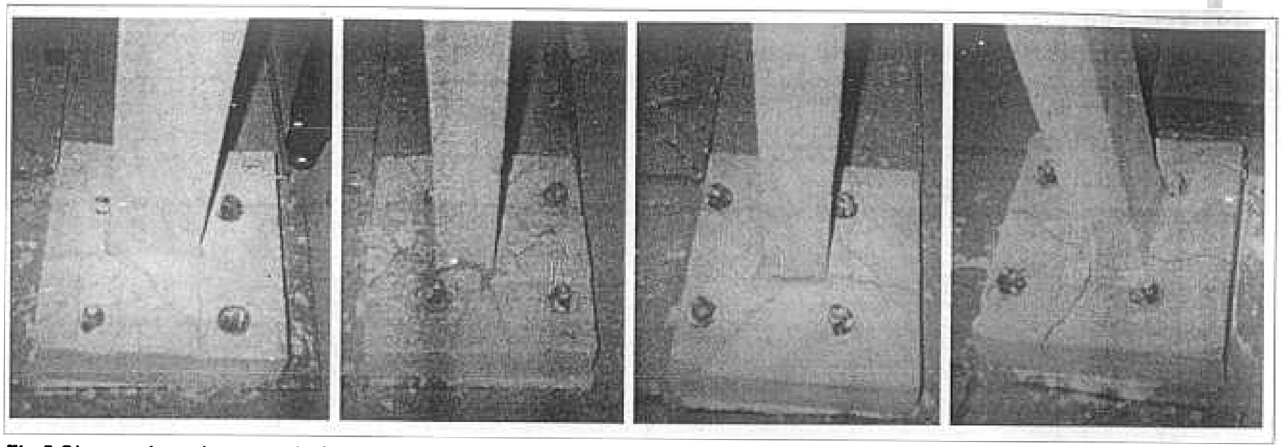

Fig 5 Observed crack pattern in footings

The test frame developed a side sway collapse mechanism after a severe cracking in the joints and the footings as shown in *Figs* 1, 4 and 5. The objective of a good and economical design is to have a collapse mechanism in which structural members realise their ultimate capacity before the joints lose their structural integrity. But contrary to this good practice, the frames failed at the joints losing their integrity by severe cracking and spalling of concrete thereby not allowing the members to reach their ultimate capacities.

Ductility

Ductility is the ability of a structural component or entire structure to undergo deformation after its initial yield without any significant reduction in its peak strength. Displacement ductility factor is the ratio of maximum displacement to the initial yield displacement. In the present test programme, the initial yield point was taken when the load-displacement curve first showed a significant amount of non-linearity following the cracking in the test structure, whereas the maximum displacement corresponded to a lateral strength which was reduced by 10 percent after reaching the peak strength. The maximum displacements attained by these frames corresponded to frame drift ratios of 4 to 5 percent, which was significantly larger than 1 to 1.5 percent storey drift ratio, typically expected in multi-storeyed frames under design earthquakes. The ductility factors attained by these frames were nearly 10, indicating special benefit from seismic detailing of IS 13920 : 1993.

Strength and stiffness

The ultimate strength of the non-seismic model was found to be 61 kN which was close to the predicted value of 56.5 kN

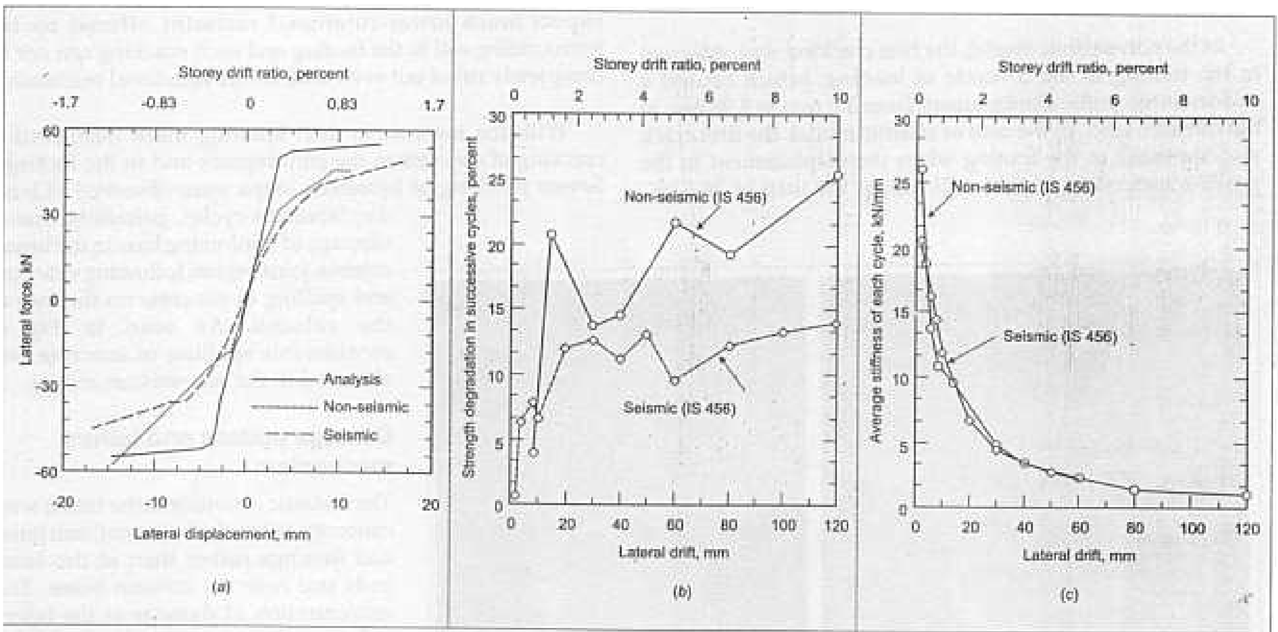

Fig 6 *(a)* Predicted and observed envelope response of frames, *(b)* strength degradation and *(c)* stiffness degradation of test frames

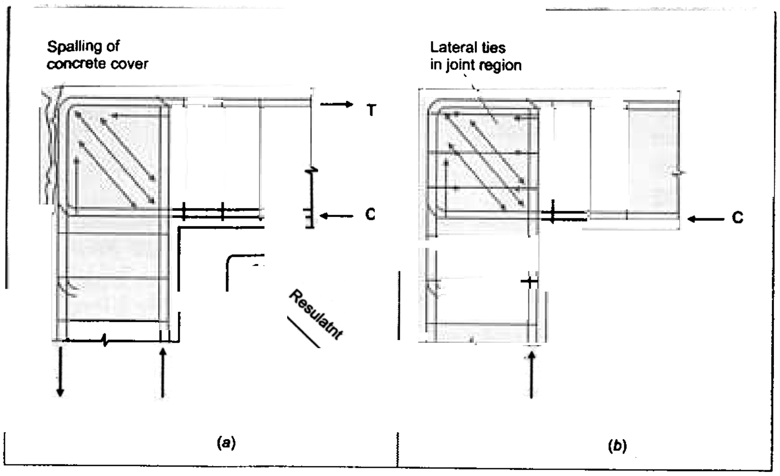

Fig 7 Action in knee beam-column joint *(a)* diagonal strut action and *(b)* lateral ties reduce loss of flexural strength of column

from the ultimate moment capacities of the various members. For the seismic model, the ultimate lateral strength was less than the non-seismic model at 53.6 kN. This lesser strength of seismic model was primarily due to the lower strength of the concrete as indicated by the compression tests on cubes. The first yield and ultimate strengths were also obtained from the SNAP-2DX non-linear analysis program which was 'nearly' matching the experimental results[11]. The yield mechanism predicted by the analysis involved the plastic hinging of beam ends and column footings. However, this expected yield mechanism could not be realised as the joints began to crack prematurely causing reduction in stiffness and strength of the frames as shown in *Fig* 6(a).

A stable strength in the hysteretic loops can be seen until first inelastic activity. The strength degradation was noted in the successive cycles of loading. It can be seen in *Fig* 6(b) that following the first yield, the strength degradation was more rapid for the non-seismic model whereas the presence of confining steel in the seismic model did help in reducing the rate of strength deterioration. The lateral stiffness of frames obtained from the test results was found less than the analytically predicted values. The theoretical stiffness was obtained by considering the joints as rigid and having full moment fixity at the base. However, the joints of the model frames were very flexible and complete moment fixity at the base could not be realised with the size of the footing provided. The contribution of joint deformation to frame deflections can be significant if the joint panel is of small volume. Furthermore, the lateral stiffness decayed apidly with increasing displacement loading as shown in *Fig 6(c)* and special detailing of IS 13920 : 1993 did not have any apparent beneficial effect on the rate of stiffness degradation.

Behaviour of joint regions

As shown in *Fig 7*, the beam-column joint resisted the joint shear predominantly due to a strut action. The longitudinal bars on the top face of the beam were bent downward into the column for the anchorage. This bending of bar supports the diagonal strut action developed to resist the joint shear. If the joint is small then it may not resist the compressive stresses developed due to the strut action and concrete in the joint region. Transverse reinforcement (lateral ties) in the joint region can decrease the stresses due to the strut action by resisting a portion of the joint shear. The beneficial influence of providing transverse reinforcement was evident from this testing programme. The lateral ties resisted severe spalling of concrete in the beam-column regions. This observation that reinforcing the frame joints with transverse reinforcement is beneficial to good earthquake performance, is also supported by past researchers[3,4]. The performance of these joints can be further improved by the actions described below.

- The spalling of concrete at the outside face of the column must be prevented and the anchorage efficiency of beam bars must be enhanced. This can be achieved by providing a beam stub equal to the development length as shown in *Fig* 8(a). The beam stub improves the joint performance in three ways: *(i)* improves anchorage conditions of beam reinforcing bars in compressive zone of the stub thus enhancing diagonal strut action, *(ii)* eliminates the possibility of concrete spalling on the outside column face thus avoiding the loss of flexural strength of the column and *(iii)* reduces congestion of reinforcement in the joint core as beam bars are anchored in the stub instead of joint region.

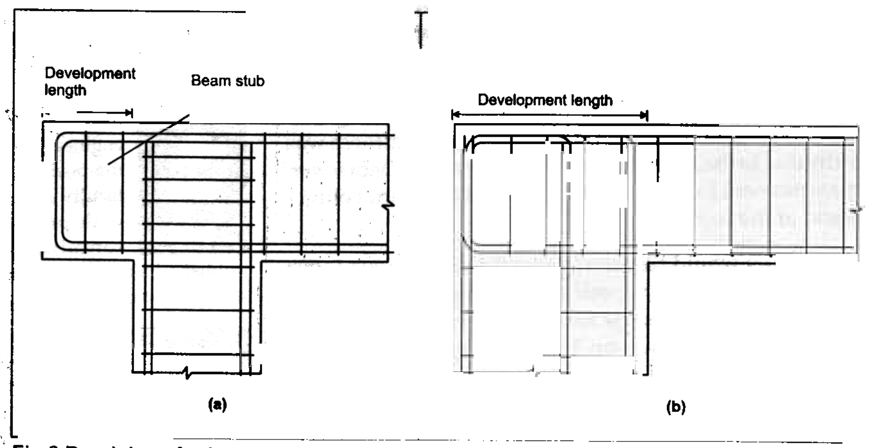

Fig 8 Provisions for improving joint strength *(a)* beam stub and *(b)* increasing column depth to beam bar anchorage

- Another alternative is to increase the depth of the column so that the complete anchorage of straight portion of beam bars can be achieved in the column itself, which is also a recommended practice of the ACI:318-99 as shown in *Fig* 8(b)[12]. Increasing column depth has major beneficial effects on joint behaviour by reducing the joint shear stress and bond stress for anchoring beam bars. This also helps in keeping flexural capacities of columns above those of beams, which is essential for the ductile strong-column-weak-beam yield mechanism for RC frames.

IS SP 34(S&T):1987 recognises the importance of lateral ties and recommends that the stirrups are provided in the column for full depth of the beam or alternatively special U-bars are detailed within the beam to restrain the column bars from buckling and to strengthen the concrete in compression[6]. Although these provisions are applicable to specific cases, they should be extended to all joints including knee as well as external beam-column joints.

Conclusion

Two one-third scale models of one-bay single-storey space frame were subjected to cyclic lateral loading to study the effect of seismic detailing on the hysteretic behaviour of RC frames. No significant differences between the hysteretic behaviour of models with non-seismic (IS 456 : 2000) and seismic (IS 3920 : 1993) details were observed because beams and columns could not develop their flexural capacities due to premature failure of joints. The test frames eventually developed a side sway collapse mechanism due to severe cracking at the beam-column joints and at the column-footing intersections. Lateral ties in joints of the seismic frame delayed the cracking and strength degradation but could not prevent the shear failure of joints. Spalling of concrete on column face must be prevented and anchorage efficiency of beam bars must be enhanced for such knee joints. Beam stubs and deep columns are two methods to achieve desired yield mechanisms.

Despite undesirable yield mechanism, a stable hysteretic behaviour for both models was observed until displacements became as large as 5 percent drift ratio. Severe pinching of the hysteretic loops was observed at large displacement cycles due to slip of beam reinforcing bars which were bent down into the column. The experimental lateral stiffness of the test structures was much lower than the computed stiffness which included the bending flexibility of the members only and considered the joint panels to be rigid. This low stiffness was primarily due to the flexible joints of small volume. Moreover, complete moment fixity was not achieved at the beam-column joints and at the footings.

The authors would like to suggest that the IS 13920:1993 which does not have any specific design recommendations for joints, should address the following issues pertaining to the joint design: *(i)* limit on the horizontal joint shear, *(ii)* anchorage and bond behaviour of beam bars in joint regions, *(iii)* joint reinforcement requirement in form of lateral ties for both horizontal and vertical shear, especially for knee joints, and *(iv)* limit on the excessive joint deformations.

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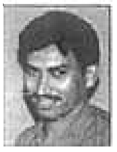

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