

## **SEISMIC RETROFIT OF BEAMS IN BUILDINGS FOR FLEXURE USING CONCRETE JACKET**

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### **ABSTRACT**

To prevent disaster in future earthquakes, one way of retrofitting the members in reinforced concrete buildings is concrete jacketing. The present study investigates the effect of jacketing on the flexural strength and performance of beams. First, slant shear tests are conducted to study the interface between old and new concrete. Second, beam specimens are tested to study the effect of jacketing on the positive bending of the span region. Third, beam-column-joint sub-assembly specimens are tested to study the effect of jacketing on the positive bending of the beams adjacent to the joint. Further, analytical investigations are carried out to predict the experimental results. A layered approach is used for the prediction of the moment versus rotation curves for the retrofitted beam specimens. An incremental nonlinear analysis is adopted to predict the lateral load versus displacement behaviour for the retrofitted sub-assembly specimens. Finally, guidelines are provided for the retrofitting of beams by concrete jacketing.

**KEYWORDS:** Beam-Column-Joint Sub-assembly, Beam, Concrete Jacketing, Layered Analysis, Retrofit, Seismic Force

### **INTRODUCTION**

Recent earthquakes have exposed the vulnerability of reinforced concrete (RC) buildings. The earthquake at Bhuj, Gujarat in 2001 has been a watershed event in the earthquake engineering practice in India. The Indian code of practice for seismic analysis has been revised to reflect the increased seismic demand in many parts of the country. Many existing buildings lack the seismic strength and detailing requirements of the current codes of practice, because they were built prior to the implementation of these codes or due to limited technical expertise. Most recently, such deficiencies were observed in Sikkim after the earthquake at the Indo-Nepal border on 18th September, 2011.

The deficiencies in the buildings, which collapsed or were damaged during the Bhuj earthquake, were reported by Murty et al. (2002). A few other observations were made at the essential facilities in the Andaman Islands after the earthquake in 2004. The common deficiencies specific to beams in RC buildings include the following:

- Inadequate positive bending moment capacity adjacent to the beam-to-column joints. The longitudinal reinforcing bars (or rebars) at the bottom of a beam tend to be discontinuous or spliced at an interior joint. For an exterior joint, the bottom bars may not be bent properly with adequate hook length. The pull-out of the bars (especially under low axial load in the column) leads to a reduced capacity and deformability of the beam (see Figure 1).
- Inadequate amount and spacing of stirrups in the potential plastic hinge regions, leading to reduced shear capacity and ductility. The stirrups may not be designed to resist the shear corresponding to the development of the beam flexural capacity. The stirrups are generally open, unless the beam is designed for torsion. The stirrups may also be warped (i.e., not in a plane after bending the bar manually) and may lack adequate end hooks. These factors lead to an absence of confinement of the core concrete, especially in the absence of a slab on the side under compression.
- Short beams in narrow bays and transfer cantilevers (provided to support the floating columns above, which do not continue to the foundation) tend to fail in shear during an earthquake.
- Lower concrete strength than the design strength and concrete of poor quality with inadequate compaction lead to weakness in the plastic-hinge regions.

To prevent any disaster during the future earthquakes, the existing deficient buildings need to be retrofitted. The selection of an appropriate retrofit scheme is based on the seismic evaluation of a building and the available resources. For a building, a combination of retrofit strategies may be selected under a retrofit scheme.



Fig. 1 Failure in a beam due to pull-out of bottom reinforcing bars (at New Passenger Terminal, Port Blair, Andaman Islands, after the earthquake in 2004)

Various retrofit strategies may be broadly classified as local and global strategies. The global retrofit strategies are applied to improve the overall behaviour of the building. In addition to a global retrofit strategy, it may be necessary to also retrofit some members locally. One way of retrofitting the beams is by concrete jacketing. This involves placing an additional layer of concrete around the existing beam, together with additional longitudinal bars and stirrups, to enhance the flexural and/or shear capacities. The present paper reports an investigation into the strengthening of beams with bottom bars discontinuous at the joints. The objective of the study is to enhance the positive (or sagging) flexural capacity of the beams at the face of the joints by introducing additional continuous bottom bars. This study has been undertaken along with the investigation by Kaliyaperumal and Sengupta (2009) on strengthening of columns.

The other options of retrofitting a beam are the fibre-reinforced polymer (FRP) wrapping and the use of steel brackets or haunches. The application of FRP wraps is convenient and the increase in the size of the beam is minimal. The positive flexural capacity of a beam at the face of the joint can be increased based on the generated confinement of concrete and the pattern and strength of the wraps. However, the increase in capacity as well as the corresponding ductility can be limited. The deficiency due to the discontinuity of the bottom bars cannot be totally rectified. On the other hand, the use of steel brackets or haunches aims to shift the potential plastic hinge region in a beam away from the face of the joint. Although these schemes are effective, the use of exposed steel may be restricted based on the corrosion, fire and architectural considerations. In the present study, concrete jacketing is investigated as a viable option of introducing continuity of the bottom bars in a beam through the joint. This investigation is aimed to improve the positive flexural capacity of the beam at the face of the joint, along with retaining substantial ductility.

For an effective concrete jacket, the use of closed stirrups as dowels involves closely spaced drilling of the existing beam. If the latter is of low strength or poor quality or small width, the drilling may damage the member. Also, drilling may intercept the existing bars unless executed carefully with a rebar locator, such as a profometer. Hence, to minimize drilling, the use of closed stirrups is precluded and only preparation of the surface of the existing concrete is investigated in the present study on retrofit for flexure.

First, slant shear tests are carried out to check the interface bond between the existing (or old) and new concrete. From these results, it is found that the specimens with roughened surface of the existing concrete and without any bonding agent fail at higher loads compared to the other specimens. Thus, it is decided that for the retrofitted specimens, the faces of the existing concrete will be roughened by using a motorised wire brush and that bonding agent will not be used.

Second, simply supported flanged beam specimens are tested under the two-point monotonic and slow cyclic loadings to study the constructability and effectiveness of the selected jacketing scheme to enhance the positive flexural capacity in the span. However, the testing of such beam specimens does not

demonstrate their behaviour in frames near the joints under the seismic loads, which are dynamic and reversed cyclic in nature. The degradation of strength and stiffness of the beams, the deterioration of bond between the rebar and concrete especially near the joints, and the subsequent pull-out of the bottom bars cannot be simulated in the beam tests. Hence, beam–column–joint sub-assemblages are tested to study the effect of jacketing of the beams under positive bending near the joints.

In the conventional mechanics of reinforced concrete, a beam is analysed or designed by using the working stress method or the limit state method (including the strength method). However, for analysing and designing for seismic forces, a performance based approach is preferred. In the latter approach, not only the strength of a structural component is evaluated, but also the load versus deformation behaviour is quantified. Hence, in the present study an analysis method suitable for heterogeneous sections is adopted to predict the behaviour of jacketed sections under a monotonic loading.

## LITERATURE REVIEW

The general advantages of concrete jacketing in the context of construction in India are as follows (Chakrabarti et al., 2008):

- Jacketing by concrete can increase both flexural and shear capacities of a beam.
- The compatibility of deformation between the existing and new concrete, resistance against delamination, and durability are better as compared to a new material on a different substrate.
- Availability of personnel skilled in concrete construction.
- The analysis of retrofitted sections follows the principles of analysis of RC sections.

Of course, there are certain disadvantages of concrete jacketing, depending on the structure and its use:

- Increase in the size and weight of the beam.
- Anchoring of bars involves drilling of holes in the existing members.
- Manufacturing of sufficiently workable concrete for the jacket.
- Possibility of disruption to the users of the building.

Despite these disadvantages, concrete jacketing is a practical option for the buildings where beams are highly deficient in flexure or shear as compared to the required demands.

Liew and Cheong (1991) tested the simply supported beams retrofitted with jackets by using pre-packed aggregate concrete. The additional reinforcement cage was attached by fixing the stirrups in the pre-located recesses. They concluded that the flexural strength can be predicted by assuming full bond between the existing and new concrete.

The experiments conducted by Alcocer and Jirsa (1993) on beam–column–slab sub-assemblages showed that concrete jacketing is effective in strengthening frame members including the joint region. A steel cage was provided to confine the joint core, which was effective up to a storey drift of 4%. However, the retrofitting scheme involved substantial drilling of the existing concrete.

Bhedaogaonkar and Wadekar (1995) retrofitted beams by using the expanded wire mesh reinforcement, while the supported slab was stiffened with an overlay. Cheong and MacAlevey (2000) conducted static and dynamic tests on 13 jacketed simply supported and continuous beams. The anchorage of bars was examined and negligible difference was noted, when the surface of the existing concrete was ‘partially roughened’ or ‘fully roughened’.

Shehata et al. (2009) strengthened beams by attaching the additional reinforcement using expansion bolts as shear connectors. From the test results, it was concluded that this scheme is efficient in increasing the flexural strength and stiffness of the beams.

## TESTS OF BEAMS

### 1. Test Setup

Simply supported beam specimens are tested under a two-point loading to study the effectiveness of concrete jacketing in increasing the mid-span positive flexural capacity. The tests are also helpful in gauging the constructability of the jacketing scheme. The test setup is shown in Figure 2. An actuator is used to apply displacement controlled loading. The applied load is measured by a load cell. Linear variable differential transducers (LVDTs) are used to measure the deformations over the gauge length equal to 250 mm, i.e., the effective depth of the reference beam. This is consistent with the assumed

length of plastic hinge for the beams in the subsequent sub-assembly specimens. Two sets of LVDTs are used on each side of the mid-span of a beam. The rotations are calculated from the deformation readings. A data acquisition system is used to record the load cell and LVDT readings.

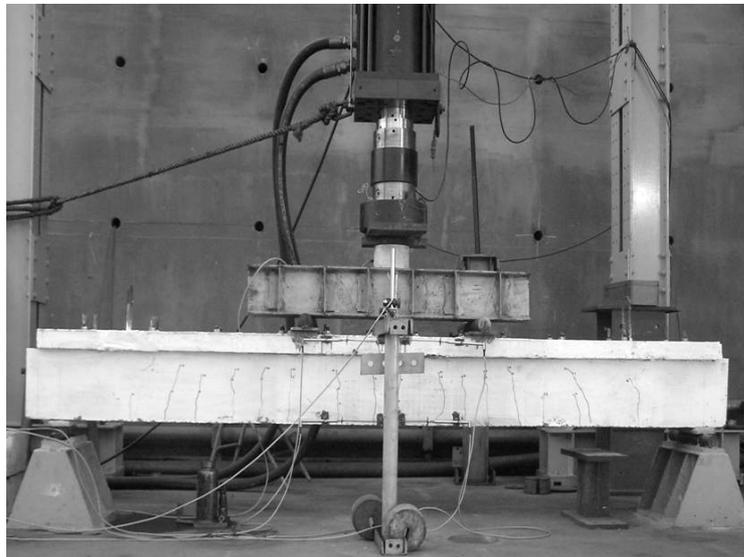
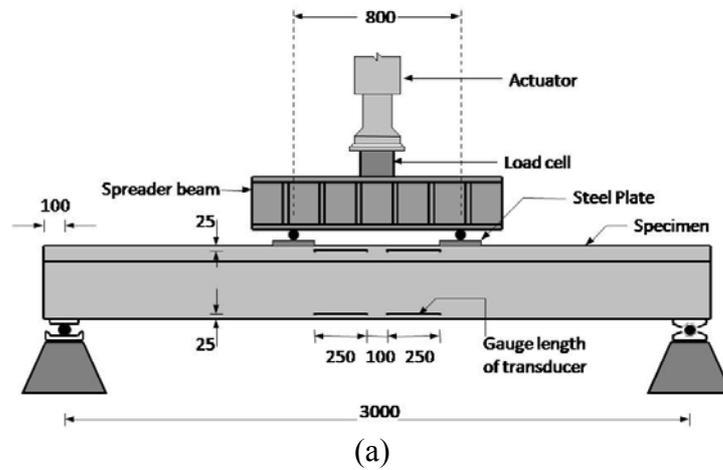


Fig. 2 Setup for testing beam specimens: (a) schematic diagram (all dimensions are in mm); (b) photograph of test setup

## 2. Specimen Details

### 2.1 Reference Specimen

A reference specimen without jacketing is tested under monotonic loading. The presence of the slab in cast-in-place beam-and-slab construction precludes the use of conventional closed stirrups. A flanged beam section is selected to simulate the obstruction due to the slab. Of course, the slab had a strengthening effect on the beams.

The sectional details of the specimen are shown in Figure 3. The material properties and reinforcement details are given in Table 1. The clear cover to the reinforcement is selected as 25 mm, since durability is not a concern in the design. The longitudinal bars are not curtailed. The stirrups at the middle third of the span are provided at a closer spacing to confine the concrete for the development of flexural plastic hinge. The concrete mix used is 1:2.35:4.65 by weight of cement, sand and coarse aggregate (of 20-mm maximum size). The water-to-cement ratio is 0.55.

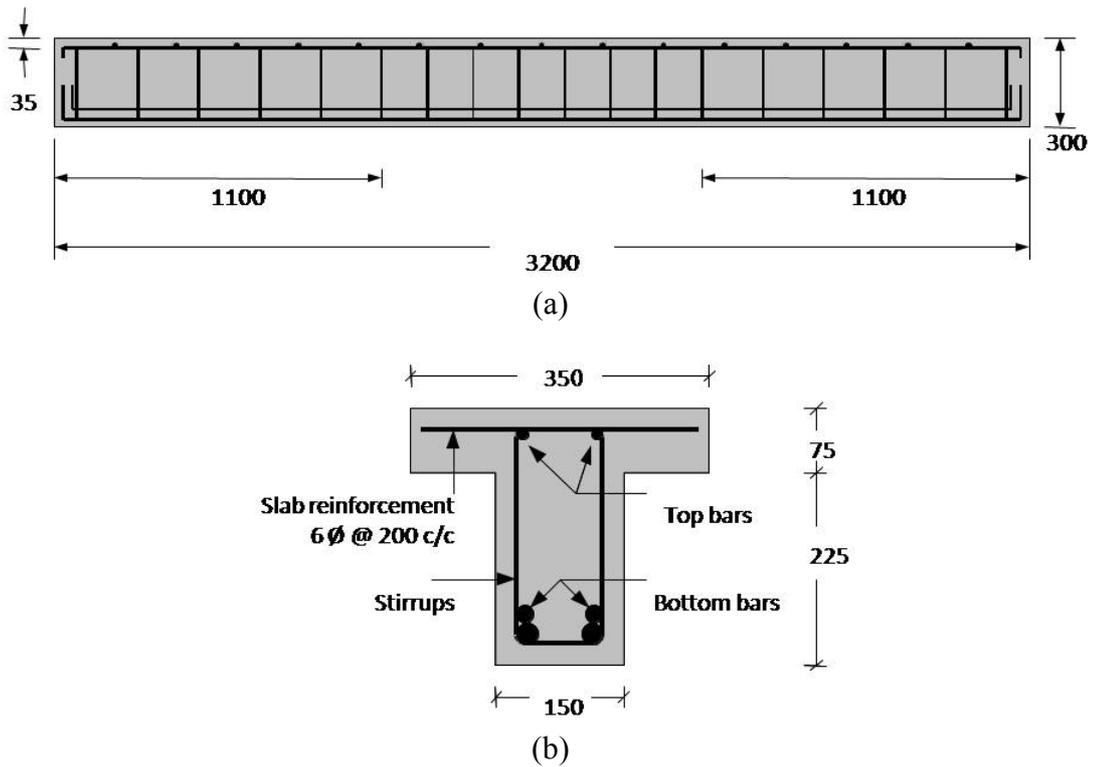


Fig. 3 Sectional details for reference beam specimen (all dimensions are in mm): (a) sectional elevation; (b) cross-section

Table 1: Material Properties and Reinforcement Details for Beam Specimens: (a) Reference Specimen; (b) Retrofitted Specimens

| (a)   |            |            |  |   |   |                 |
|---|------------|------------|--|---|---|-----------------|
| Specimen  | Concrete   |            | Reinforcement  |   | Loading   |                 |
|   | $f_{cm,E}$ |            | Longitudinal Bars  | $f_y$   |   | Transverse Bars |
| 1   | 24         |            | Top: 2-16 $\phi$<br>Bottom:<br>2-16 $\phi$ +<br>2-20 $\phi$<br>( $p = 1.7\%$ ) | 416<br>416<br>420   | 2 legged, 10 $\phi$<br>@ 150 c/c in the middle<br>third of the span, and<br>@ 175 c/c at the ends | Monotonic       |
| (b)   |            |            |  |   |   |                 |
| Specimen  | Concrete   |            | Additional Reinforcement*  |   | Loading   |                 |
|   | $f_{cm,E}$ | $f_{cm,J}$ | Longitudinal Bottom Bars   | Transverse Bars   |   |                 |
| 1   | 20         | 25         | 2-16 $\phi$<br>(resultant $p = 1.5\%$ )  | 2 legged, 8 $\phi$<br>@ 100 c/c in the middle<br>third of the span<br>@ 150 c/c at the ends | Monotonic   |                 |
| 2   | 17         | 33         |  |   | Cyclic  |                 |
| $f_{cm,E}$ = mean cube strength of existing concrete in MPa<br>$f_{cm,J}$ = mean cube strength of jacket concrete in MPa<br>$f_y$ = yield strength of steel in MPa<br>$\phi$ = diameter of bars in mm<br>$E$ = modulus of elasticity for steel (i.e., $2 \times 10^5$ N/mm <sup>2</sup> )<br>$p$ = percentage of bottom steel |            |            |  |   |   |                 |

\*These bars are in addition to those in the existing section. The existing section has the same reinforcement as the reference specimen.

## 2.2 Retrofitted Specimens

Two retrofitted specimens are tested, one under each of the monotonic and half-cyclic loadings. For a retrofitted specimen, the cross-section, reinforcement detailing and the concrete mix of the inner (i.e., existing) portion are similar to the reference specimen. In jacketing a beam, the additional longitudinal bars should be limited so as to retain the ductile behaviour of the under-reinforced section. The sectional details of the specimens are shown in Figure 4. The material properties and reinforcement details are given in Table 1.

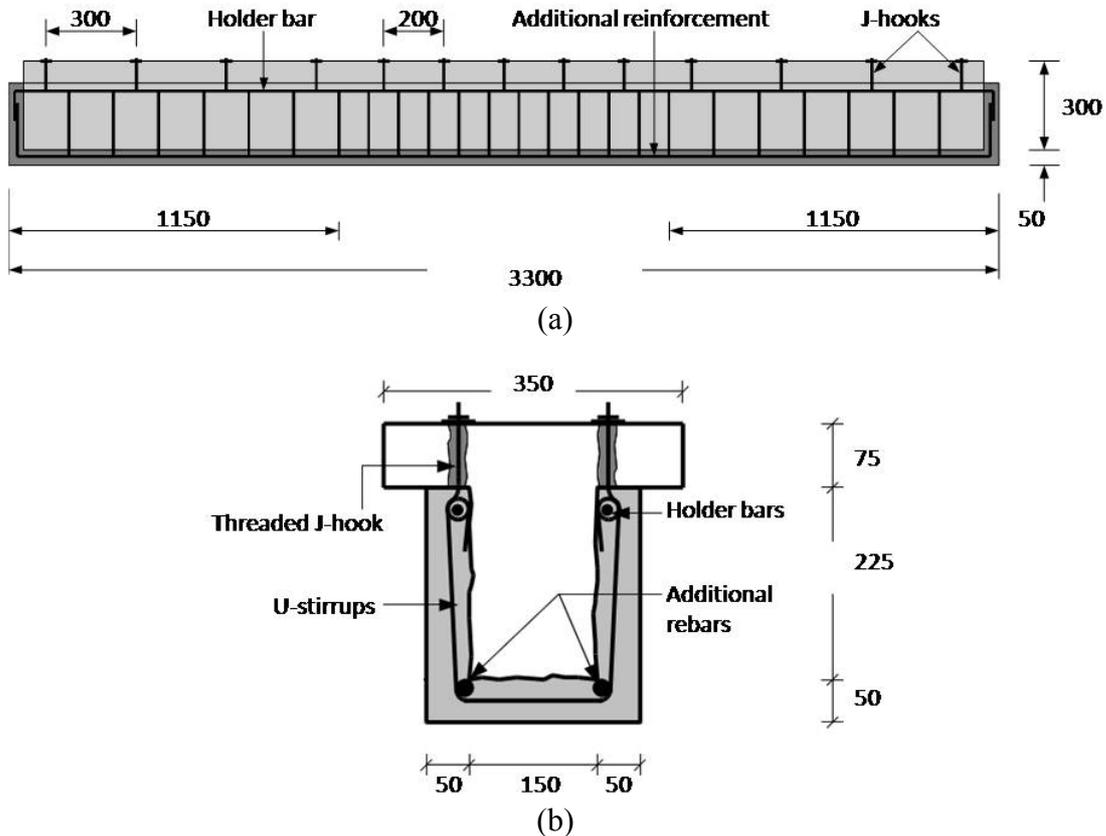


Fig. 4 Sectional details for retrofitted beam specimens (all dimensions are in mm): (a) sectional elevation; (b) cross-section

For ensuring the composite action of the jacketed beams, first the surface of the existing concrete is roughened with a motorised wire brush. The surface is cleaned with a jet of water to remove the dust. Next, the additional reinforcement cage for the jacket is anchored to the existing beam. A seismic retrofit requires closely spaced stirrups for the confinement of concrete. Drilling the slab or beam at the interval of closed stirrups can lead to considerable damage due to cracking and pop-outs. Moreover, drilling of the beam is not recommended, if the existing concrete is of poor quality. Hence, the reinforcement cage is made of open stirrups and attached to the slab by J-hooks, with holes drilled at a larger interval—in this case, at double the spacing of stirrups. The zone of compression in the beam spreads into the slab, which acts as a flange. This reduces the deficiency due to the lack of closed stirrups by delaying the crushing of concrete. The size of the J-hooks is selected such that the strength of the hooks per unit length is larger than the yield strength of the stirrups. Based on the availability, the 20-mm diameter J-hooks made of mild steel are used. Of course, the J-hooks made of high-strength steel are preferable.

The sequence of the fabrication and installation of the rebar cage is as follows:

1. Two holder bars, one for each side of the beam are passed through the eyes of two sets of J-hooks.
2. The stirrup bars are bent into the U-shape with hooked ends.
3. The stirrups are hung and secured by binding wires to the holder bars at the required spacing.
4. The required number of longitudinal bars is tied to the stirrups at the bottom corners.
5. Holes are drilled in the slab as per the spacing of the J-hooks.
6. The fabricated cage is lifted to the underside of the existing beam.

7. The threaded ends of the J-hooks are inserted through the holes drilled in the slab and are secured with nuts.

With the above technique, the reinforcement cage can be fabricated separately and then attached to the underside of an existing beam. This is convenient when applied to an existing building with high ceiling. Moreover, the technique is applicable without damaging any wall above the beam. Of course, if there is a wall beneath the beam, a few courses of masonry have to be removed to accommodate the cage. A photograph of the installed cage is shown in Figure 5.

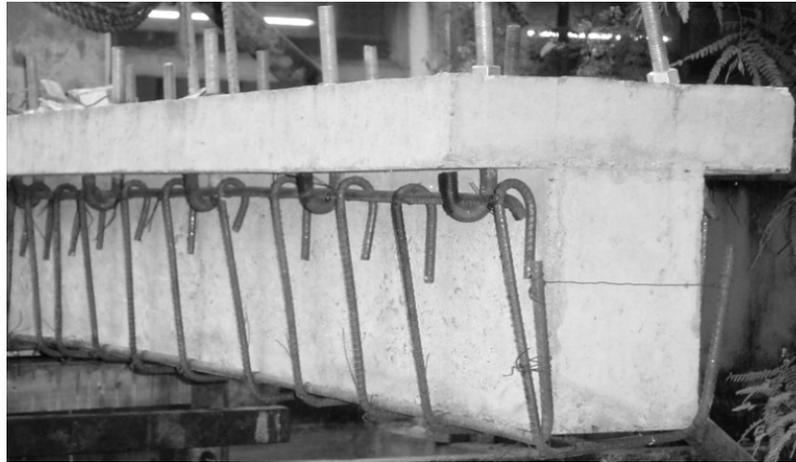


Fig. 5 Reinforcement cage for concrete jacket after installation

The jackets are made of self-compacting concrete (SCC). After several trials, a design mix of the SCC is formulated and the flowability is tested by slump flow test. Segregation of the concrete is not observed in the test.

### 3. Test Results

#### 3.1 Reference Specimen

The behaviour of the specimen is found to be typical of an under-reinforced beam. The failure is characterized by well-spaced cracks in the soffit of the beam. The yielding of reinforcement is inferred from the opening of cracks and from the strain gauge readings. At each step, the rotation is calculated from the LVDT readings. An average of the two values of rotation from the two sets of LVDTs is considered.

#### 3.2 Retrofitted Specimens

Under the monotonic loading, the behaviour of the retrofitted specimen is found to be similar to that of the reference specimen. Well-distributed flexural cracks are observed initially. Close to failure, the significant cracks are found to be localized in the middle-third region. The bottom reinforcement yields before the crushing of concrete at the top. There is no apparent delamination of the jacket up to the peak load. This is checked by light tapping with a hammer, which would reveal an air gap. Figure 6 shows the moment versus rotation curves for the reference and retrofitted specimens. The retrofitted specimen is found to have higher moment capacity with comparable ductility with respect to the reference specimen.

For the half-cyclic loading, at each displacement level, five cycles of loading are applied to study the strength and stiffness degradations. The increment of the displacement levels is taken to be the theoretical displacement corresponding to the yielding of the additional longitudinal bars. The behaviour of the specimen is characterized by a well-distributed cracking in the initial stages of loading. However, with the increasing number of cycles of loading, flexural shear cracks are observed to develop in the outer regions, in addition to the flexural cracks in the middle-third region (see Figure 7). A study of the variation of peak moment for each cycle of loading shows that the degradation of strength with the load cycles is gradual, even after reaching the flexural capacity.

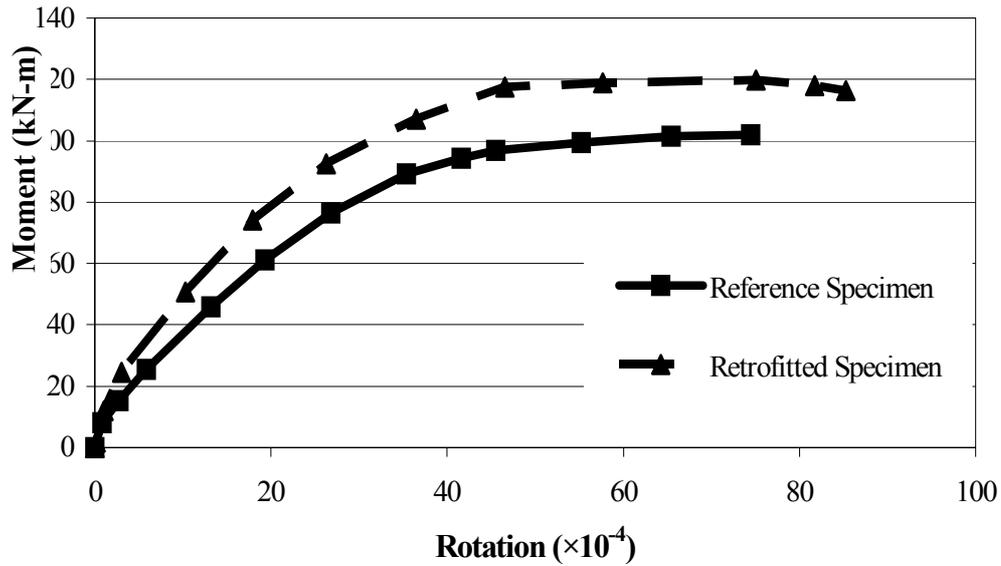


Fig. 6 Comparison of moment versus rotation curves for beam specimens tested under monotonic loading

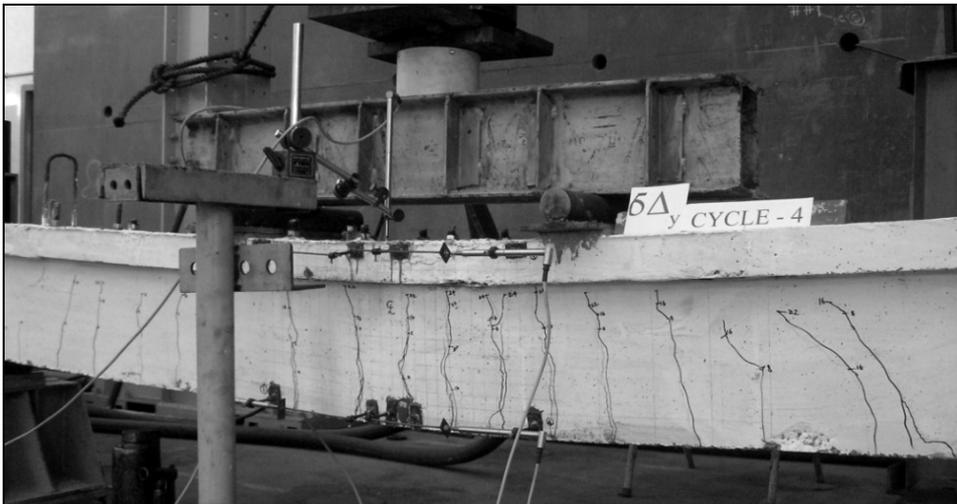


Fig. 7 Retrofitted beam specimen during testing under half-cyclic loading

#### TESTS OF BEAM-COLUMN-JOINT SUB-ASSEMBLAGES

The testing of the sub-assemblages is explained in an earlier publication (Kaliyaperumal and Sengupta, 2009). However, a brief description is provided here for ready reference.

The sub-assemblage specimens were tested by using a facility that consisted of a reaction wall, a strong floor and a test frame (see Figure 8). The frame was designed and fabricated as under the present study. A specimen was erected in a steel assembly at the bottom, which had the provision of placing a hydraulic jack (for applying an upward vertical load at the bottom end of the lower column) and a vertical sliding-cum-rocking pin support. The top end of the upper column was attached to the frame through a spacer assembly and a horizontal sliding-cum-rocking bearing assembly. The lateral load was applied at the top end by an actuator fitted to the adjacent reaction wall. The ends of the beams were supported on the horizontal sliding-cum-rocking roller bearings placed on pedestals. The hold-down steel members restricted any uplift of the ends of the beams. The loads were measured by using load cells. The deformations of the beams were measured by using LVDTs near the joint, while considering a plastic hinge length equal to the effective depth of the beams of the reference specimens. The details of the test facility developed as part of this research are provided in some earlier publications (Badari Narayanan, 2010; Sengupta et al., 2006).

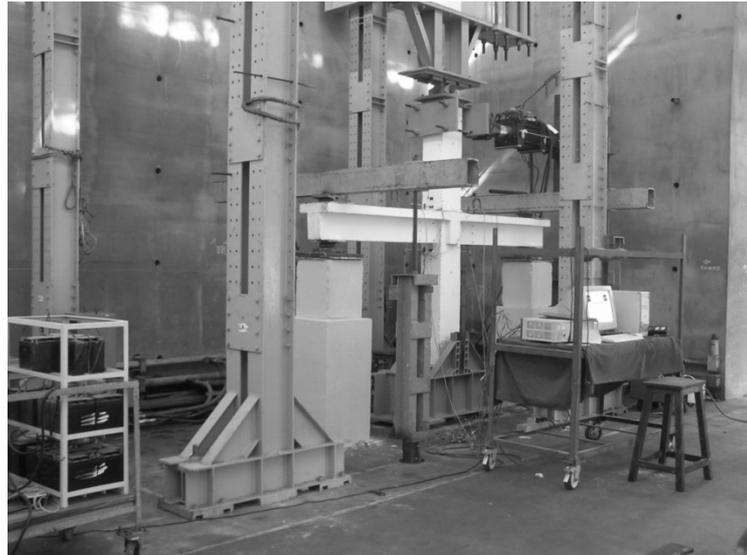


Fig. 8 Setup for testing sub-assembly specimens

To study the lateral load versus displacement behaviour, two reference and two retrofitted specimens are tested. For each type, one specimen is tested under the monotonic lateral load and the other under the cyclic lateral load. The displacement-controlled cyclic load history is shown in Figure 9. At each displacement level, three cycles of loading are applied.

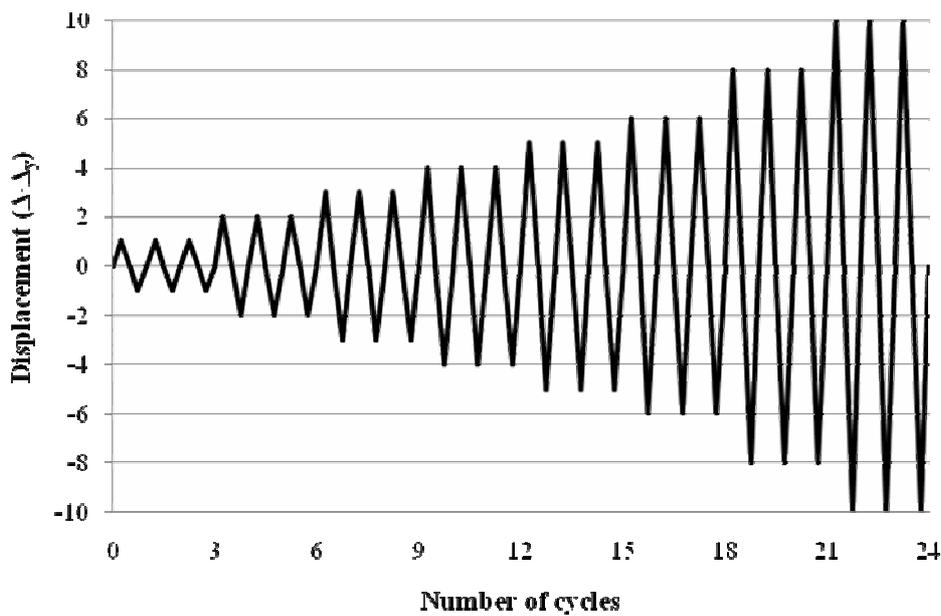


Fig. 9 Cyclic loading history

### 1. Specimen Details

The objective of the present study is to investigate the effects of jacketing on the positive flexural capacity of beams at the face of an interior joint. Hence, failure of the columns and shear failures of the beams or the joints are deliberately avoided. To avoid the formation of a plastic hinge in the columns prior to that in the beams, moment capacity of the columns is kept substantially higher than the moment capacity of the beams after retrofitting.

Two reference and two retrofitted specimens are tested. For each type, one specimen is tested under the monotonic and the other under the cyclic loading. The height of the specimen from the bottom pin support to the rocker pin at the top is 2.1 m. The total length of the beams between the centre-lines of the left and the right roller bearings is 3 m. Stub beams in the transverse direction and slab over the beams are

provided to create obstruction in placing the additional longitudinal and transverse bars in the jacket, like in an interior joint of an existing building.

### 1.1 Reference Specimens

The sectional details of a specimen are shown in Figure 10. Table 2 provides the material properties and reinforcement details for the specimens. The bottom reinforcing bars of the beam are deliberately discontinued at the joint, to simulate the conventional condition as per the gravity load design. The embedment length for the 8-mm diameter bars is selected as  $L_d/3 \approx 150$  mm, where  $L_d$  is the development length in compression. Further, a concrete mix of 1:2.15:3.27 (i.e., cement: fine aggregate: coarse aggregate) proportions with a water-to-cement ratio of 0.55 is used.

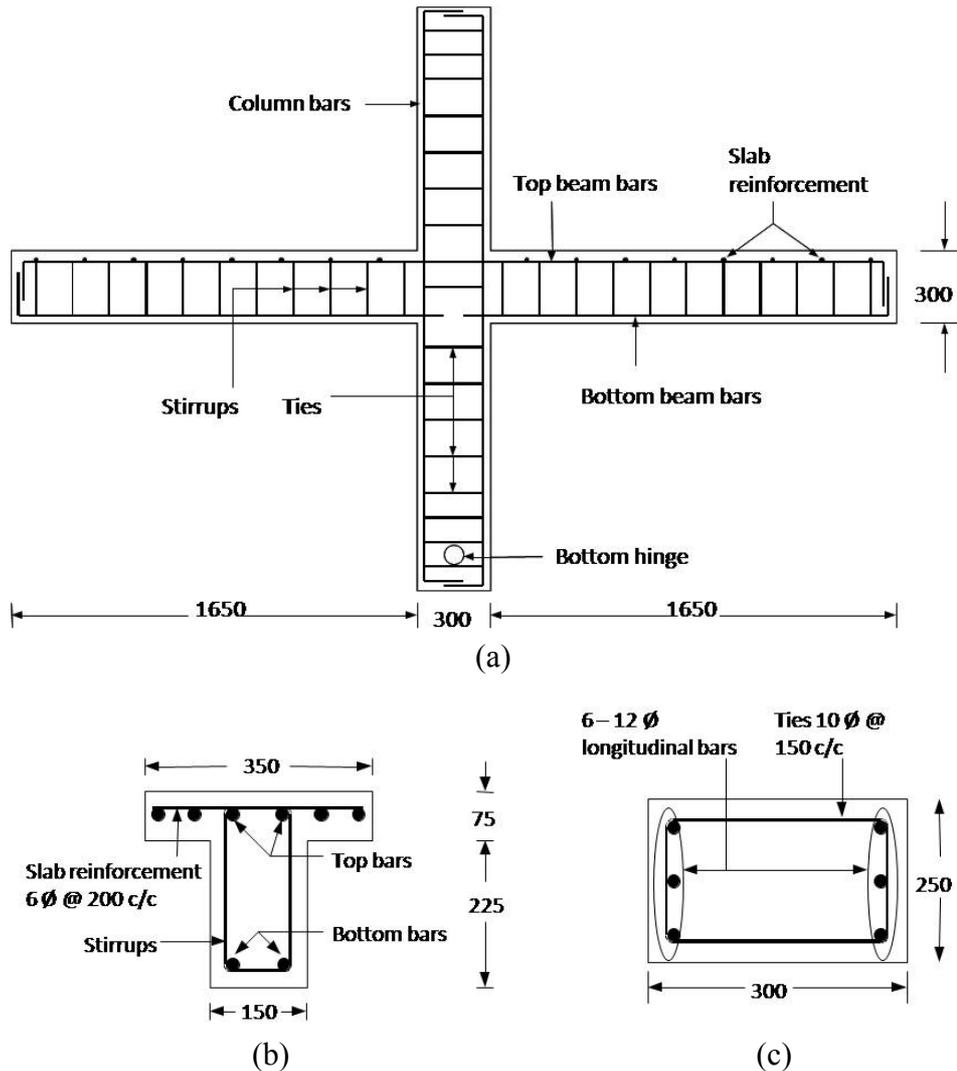


Fig. 10 Sectional details for reference sub-assembly specimens (all dimensions are in mm): (a) sectional elevation; (b) beam section; (c) column section

### 1.2 Retrofitted Specimens

The details of the specimens are shown in Figure 11. The fabrication of the reinforcement for the jacket is illustrated in Figure 12. The construction procedure used for jacketing the sub-assembly specimens is similar to that for the beam specimens. In the joint region, the additional bottom longitudinal bars are made continuous around the joint by cranking with a splay of 1 in 10. This avoids drilling through the joint. The amount of additional reinforcement is limited to ensure that the column is still stronger than the beam. The stirrups provided along the length of each splay, with spacing as per the ductile detailing requirements for a potential plastic hinge (BIS, 1993), reduce the straightening of the splay.

**Table 2: Material Properties and Reinforcement Details for Sub-assembly Specimens: (a) Reference Specimens; (b) Retrofitted Specimens**

| (a)   |            |            |  |   |   |                 |
|---|------------|------------|--|---|---|-----------------|
| Specimen  | Concrete   |            | Reinforcement                          |   | Loading   |                 |
|   | $f_{cm,E}$ |            | Longitudinal Bars                      | $f_y$   |   | Transverse Bars |
| 1   | 39         |            | Top: 6-10 $\phi$<br>Bottom: 2-8 $\phi$ | 450   | 2 legged, 8 $\phi$<br>@ 150 c/c in the middle third of the span, and<br>@ 200 c/c at the ends | Monotonic       |
| 2   | 15         |            | ( $p = 0.17\%$ )                       | 420   |   | Cyclic          |
| (b)   |            |            |  |   |   |                 |
| Specimen  | Concrete   |            | Additional Reinforcement*              |   | Loading   |                 |
|   | $f_{cm,E}$ | $f_{cm,J}$ | Longitudinal Bottom Bars               | Transverse Bars   |   |                 |
| 1   | 20         | 27         | 2-10 $\phi$                            | 2 legged, 8 $\phi$<br>@ 100 c/c in the middle third of the span | Monotonic   |                 |
| 2   | 19         | 38         |  | @ 150 c/c at the ends   | Cyclic  |                 |
| $f_{cm,E}$ = mean cube strength of existing concrete in MPa<br>$f_{cm,J}$ = mean cube strength of jacket concrete in MPa<br>$f_y$ = yield strength of steel in MPa<br>$\phi$ = diameter of bars in mm<br>$E$ = modulus of elasticity for steel (i.e., $2 \times 10^5$ N/mm <sup>2</sup> )<br>$p$ = percentage of bottom steel |            |            |  |   |   |                 |

\*These  $f_y = 450$  MPa bars are in addition to those in the existing section. The existing section has the same reinforcement as the reference specimen.

The additional longitudinal bars of the beams should be secured to the column reinforcement for the transfer of moment between the beams and columns in a rigid joint. In existing buildings, the column should also be jacketed to make it stronger than the jacketed beams and to satisfy the design force requirements. Hence, the columns in the specimens are also jacketed with additional longitudinal bars and ties. The slab is drilled near the four corners of the column and additional bars are continued through the holes. The additional longitudinal bars provided in the beams and columns are secured together with binding wires. Since the joints are provided with an adequate number of ties, additional confining of the joint during jacketing is not adopted. However, in a parallel study on the strengthening of columns, the joint has been confined by welding angle sections to the additional longitudinal bars for the columns (Kaliyaperumal and Sengupta, 2009).

The specimens are cast vertically to simulate the actual method of construction. The jackets of the retrofitted specimens are made of the self-compacting concrete. After the retrofit, the column jacket is flushed with the face of a transverse stub beam.

## 2. Test Results

For each specimen, first a constant vertical compression, approximately equal to the balanced failure load of the column in a reference specimen, is applied on the column. Next, the lateral load is applied at the top end of the upper column till the limit of the stroke length of the actuator. This is found to correspond to a drift ratio of 5.2%, where drift ratio is defined as the relative drift divided by the height between the top and bottom pins.

### 2.1 Reference Specimens

For the specimen tested under the monotonic lateral loading, the lateral displacement is found to be nearly zero till the load of about 10 kN. Thereafter, a sudden jump in the displacement is noticed at this load. It is inferred that this is due to the effect of friction generated at the sliding bearing in the presence of the vertical load. The first few cracks near the joint, at the soffit of the beam undergoing sagging and at

the slab undergoing hogging, are found to be dispersed. Close to the peak lateral load, single major cracks are observed on both sides of the joint (see Figure 13(a)). Even though the bottom longitudinal bars in the beams are discontinuous at the joint, an appreciable pull-out of the bars is not observed. This is because a small diameter of the bars generates good bond and the axial compression through the column enhances this bond. At the peak load, the lateral drift at the top is about 20 mm and the corresponding drift ratio is 0.95%.

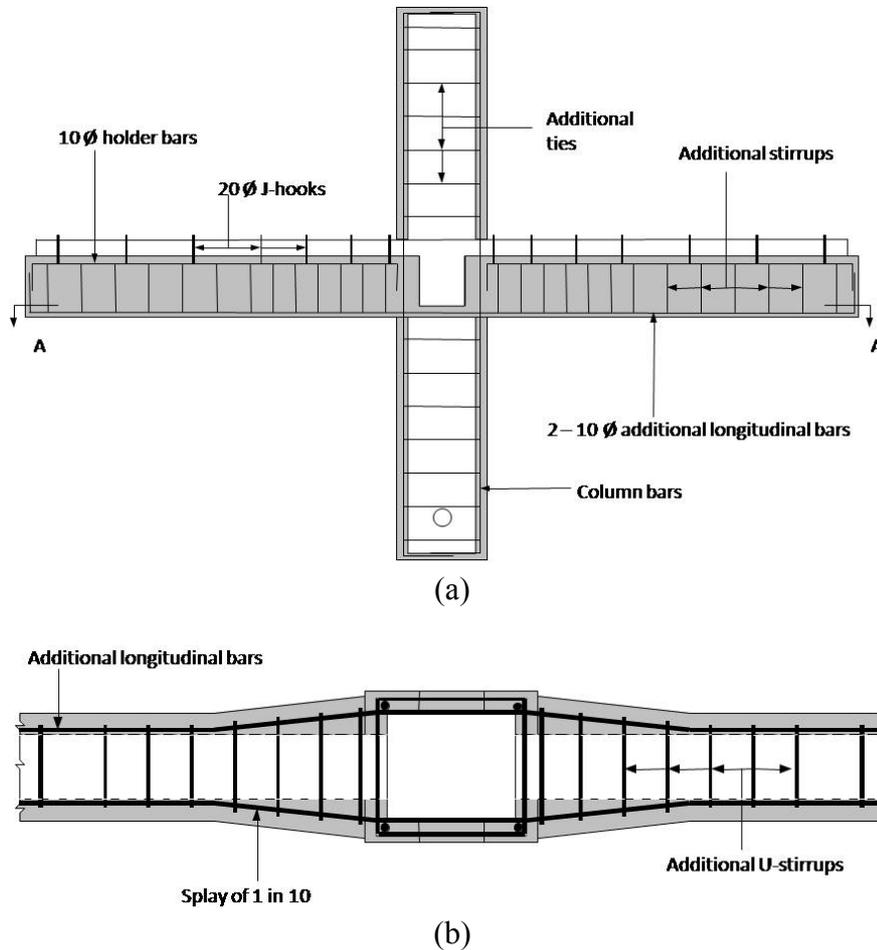


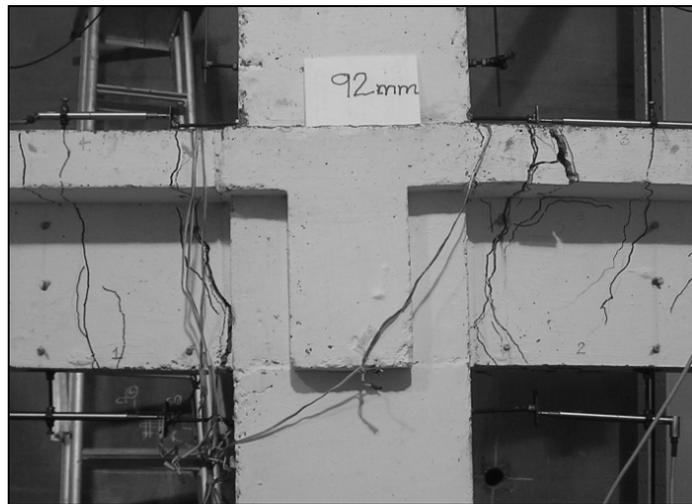
Fig. 11 Sectional details for retrofitted sub-assembly specimens: (a) sectional elevation, (b) section A-A



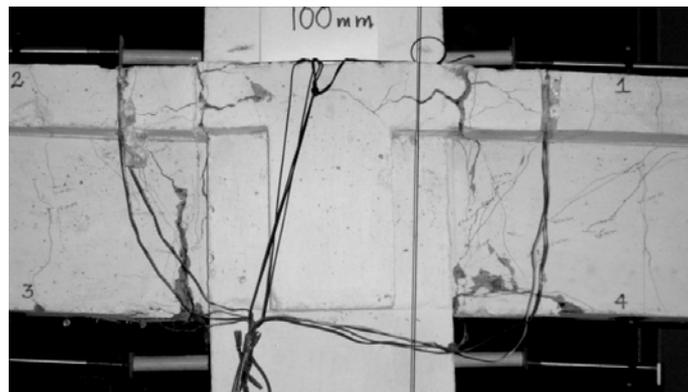
Fig. 12 Fabrication of reinforcement for retrofitted sub-assembly specimens

For the specimen tested under the cyclic lateral loading, the spalling of the cover concrete and pull-out of the bottom bars is observed after several cycles of loading. Vertical cracks are found to form on the

top of the slab due to hogging. A horizontal crack is found to propagate towards the joint at the level of the top reinforcement, thus showing the degeneration of bond and yield penetration into the joint (see Figure 13(b)).



(a)



(b)

Fig. 13 Close-up view of reference sub-assembly specimens during testing under: (a) monotonic loading; (b) cyclic loading

## 2.2 Retrofitted Specimens

The behaviour of the retrofitted specimen tested under the monotonic loading is similar to that of the reference specimen. No spalling of concrete or buckling of the additional longitudinal bars is observed throughout the experiment. The failure of beams is characterized by the formation of major cracks at the faces of the original joint (see Figure 14(a)). The crack-width is large due to the yielding of the longitudinal bars of the beams. The portion of the jacket concrete around the corners of the joint is found to be delaminated. At the peak load, the observed lateral drift at the top is about 42 mm and the corresponding drift ratio is 1.9%.

The retrofitted specimen tested under the cyclic loading is found to behave satisfactorily with regard to strength and ductility without any premature failure, such as buckling of the longitudinal bars or delamination of the jacket (see Figure 14(b)). However, the pinching effect is not found to be reduced.

The lateral load versus sagging rotation curves for the reference and retrofitted specimens tested under the monotonic loading are compared in Figure 15. The lateral load capacity of the retrofitted specimen shows improvement compared to that of the reference specimen. Although the stiffness of the retrofitted specimen is affected by the jacketing of the column, the strength is primarily governed by the attaining of the positive flexural capacity of the retrofitted beams.

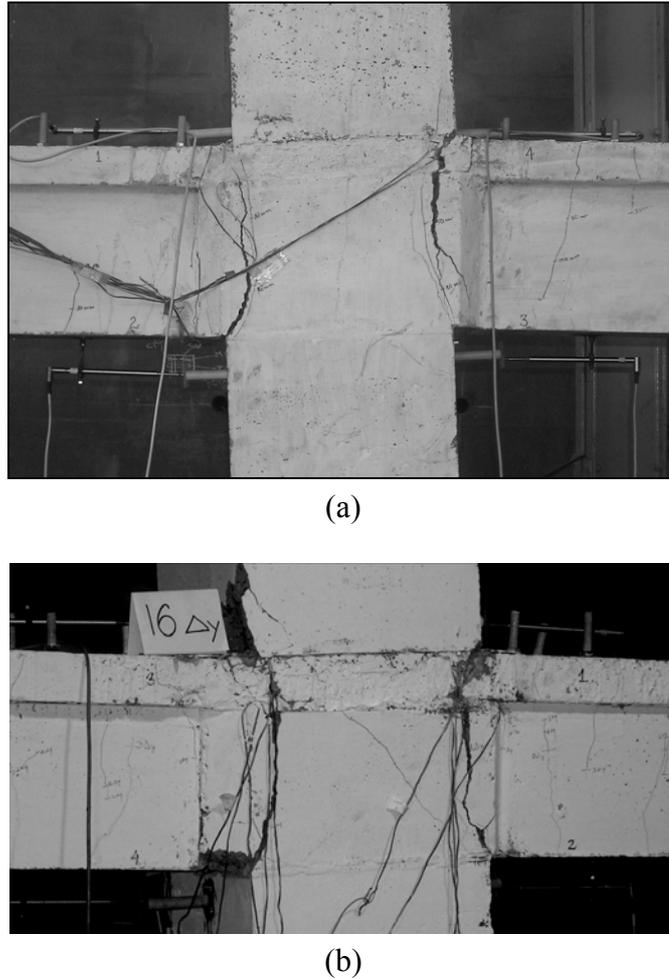


Fig. 14 Close-up view of retrofitted sub-assembly specimens during testing under: (a) monotonic loading; (b) cyclic loading

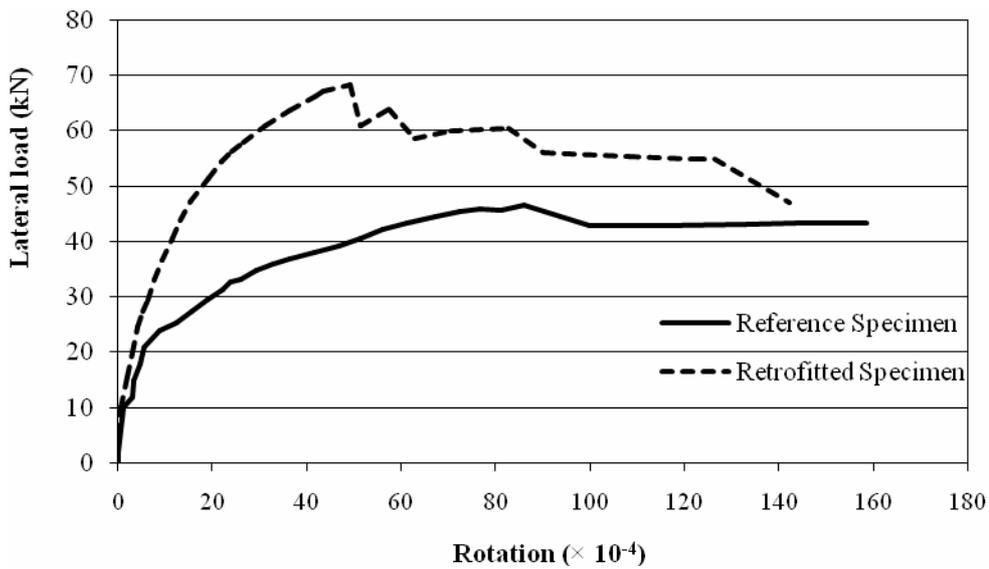


Fig. 15 Comparison of lateral load versus sagging rotation curves for the sub-assembly specimens tested under monotonic loading

The lateral load versus displacement curves for the reference and retrofitted specimens tested under the cyclic loading are compared in Figure 16. It may be observed that there is an increase in the lateral strength after retrofitting. Also, there is an increase in the energy dissipation. However, the pinching of

the hysteresis loops is not improved. This possibly happens as the yielding of longitudinal bars in the beams leads to a wide opening of cracks.

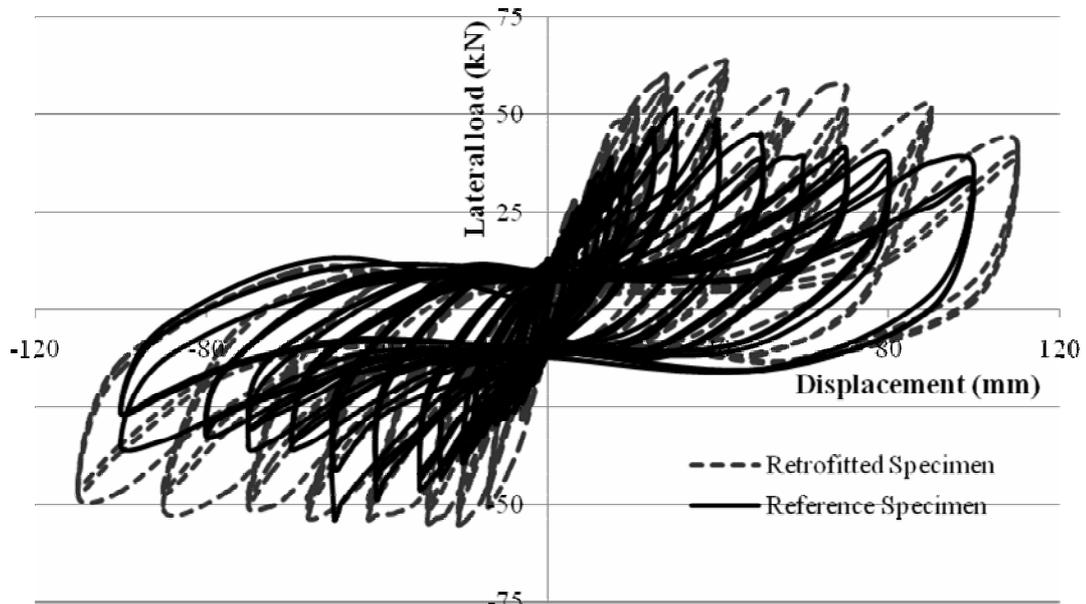


Fig. 16 Comparison of lateral load versus displacement curves for the sub-assembly specimens tested under cyclic loading

**2.3 Comparison of Monotonic and Cyclic Tests**

The lateral load versus displacement curves for the monotonic and cyclic loadings of the reference specimen are overlapped in Figure 17. The corresponding curves for the retrofitted specimens are shown in Figure 18. For both the reference and retrofitted specimens, the monotonic tests are found to provide sufficiently good estimates of the envelope behaviour of the lateral load versus displacement curves obtained under the cyclic loading. Hence, the values of strength and ductility, as calculated from the lateral load versus displacement curves obtained under the monotonic loading, may be considered to be rational estimates for the cyclic loading. Of course, energy dissipation can be estimated only from the cyclic test.

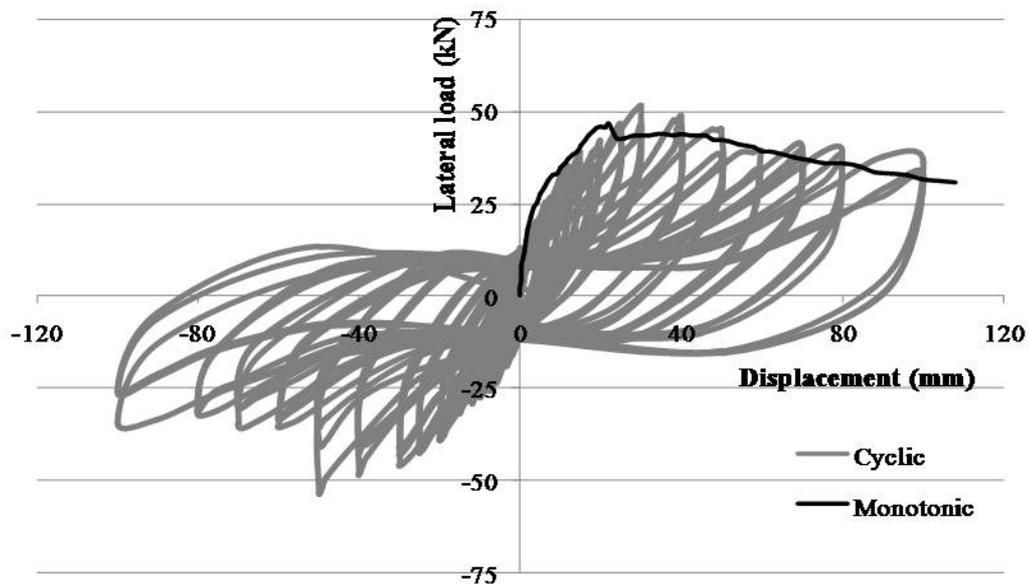


Fig. 17 Comparison of lateral load versus displacement curves for the reference sub-assembly specimens tested under monotonic and cyclic loadings

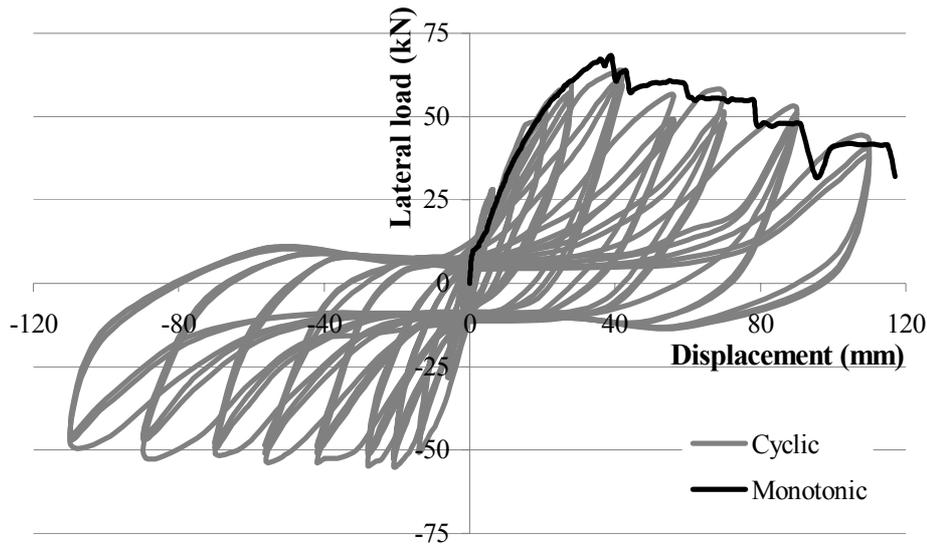


Fig. 18 Comparison of lateral load versus displacement curves for the retrofitted sub-assembly specimens tested under monotonic and cyclic loadings

#### 2.4 Observations

Following observations are made from the beam–column–joint sub-assembly tests:

1. The retrofitted specimens do not show any apparent delamination between the existing concrete and the concrete in the jacket.
2. The lateral load capacity of the retrofitted specimen tested under the monotonic loading is 1.46 times that of the reference specimen.
3. The lateral load capacity of the retrofitted specimen tested under the cyclic loading is 1.23 times that of the reference specimen.
4. Under the cyclic loading, the cumulative energy dissipated till the attaining of the peak load (at a displacement of 42 mm) for the retrofitted specimen is 14285 kN-mm, as against 12697 kN-mm for the reference specimen (at the same displacement).

#### ANALYSIS OF RETROFITTED SECTIONS

The moment versus rotation curves for the retrofitted sections are predicted by the layered (or lamellar) analysis approach. Although this approach is well established for heterogeneous sections, the objective of the present analysis is to investigate its applicability for a retrofitted section in a member under predominantly flexural deformations. A simplified analysis considering a uniform grade of concrete is compared with the rigorous analysis, as the simplified analysis may be more suitable for professional practice.

First, the behaviour of the retrofitted beam specimen tested under the monotonic loading is predicted. Next, the predicted behaviour of the beams near the joint is used to estimate the lateral load versus displacement behaviour of the retrofitted sub-assembly specimen tested under the monotonic loading.

##### 1. Layered Analysis

A retrofitted section is a heterogeneous section with two grades of concrete and several layers of reinforcement bars. To account for the heterogeneity, a layered method of analysis is used, wherein the section is divided into layers through the depth. For the concrete under compression, the parabolic-plastic stress versus strain model, as given in IS 456 (BIS, 2000), is used for the existing concrete as well as the concrete for the jacket, with the corresponding strengths as given in Tables 1 and 2. Any confinement of the existing concrete is neglected as closed stirrups are not used in the jacket. A perfect bond for the strain compatibility between the existing concrete and the jacket is assumed. For concrete under tension, the effect of tension stiffening is considered by using the stress versus strain model proposed by Kaklauskas and Ghaboussi (2001). For the steel bars, an elasto-plastic stress versus strain model is adopted. For a sub-assembly specimen, the tensile strength of the discontinuous bottom bars in a beam is reduced based on the provided embedment length.

## 2. Simplified Analysis

The simplified analysis is based on a uniform concrete strength (which is same as that of the existing concrete) throughout the section. The tension stiffening effect of concrete is neglected.

## 3. Results for the Beam Specimen

Figure 19 shows the comparison of the theoretical predictions with the test results of the moment versus rotation behaviour at the central region for the retrofitted beam tested under the monotonic loading. The test results show a higher stiffness for the beam than that predicted, in the initial stages from the cracking of concrete up to the yielding of the bars. Thus, the tension stiffening is underestimated in the layered analysis. Of course, this effect is significant only up to the yielding of the bars. Beyond that, there is a good agreement between the experimental and theoretical curves. The observed flexural capacity of the specimen is only 4% less than that predicted. The ductilities shown by the two curves are comparable. Hence, it is concluded that there is no slippage between the concrete layers, and the scheme selected for retrofitting is effective in increasing the positive flexural capacity while retaining the ductility.

There is little or no difference between the results of the layered and the simplified analyses. This is because the thickness of the jacket is small and the compressive strengths of the two grades of concrete are not widely different.

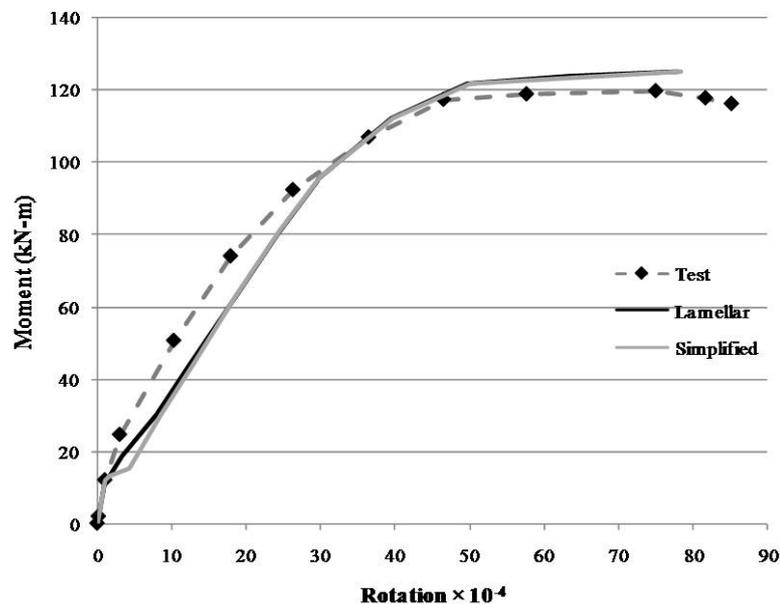


Fig. 19 Comparison of moment versus rotation curves for the retrofitted beam specimen tested under monotonic loading

## 4. Results for the Sub-assembly Specimen

The lateral load versus displacement curve for the retrofitted specimen tested under the monotonic lateral loading is predicted by a computational model of the sub-assembly. Initially, a conventional pushover analysis is conducted by assigning bilinear moment versus rotation properties (up to the peak) for the lumped plastic hinges in the beams at the faces of the joint. It is observed that the nonlinearity in the overall behaviour could not be predicted with sufficient accuracy. Next, an incremental nonlinear analysis is conducted by considering spread plasticity and by varying flexural stiffness for the plastic hinge regions of the beams. The method of analysis used here is explained in an earlier publication (Kaliyaperumal and Sengupta, 2009). Only a brief description is provided below for ready reference.

The computational model for the sub-assembly developed by using frame elements and appropriate boundary conditions is shown in Figure 20. To consider spread plasticity, the beam members are subdivided for isolating the plastic hinge regions of length 250 mm (which is approximately equal to the effective depths of the beams) from the faces of the joint. Next, at each step of the lateral load  $P_{Hi}$ , the secant flexural stiffness values, based on the predicted moment versus rotation curves from a layered analysis of the beam section under sagging and hogging, are assigned to the plastic-hinge elements. Next,

a linear analysis is performed to get the lateral displacement  $\Delta_i$  at the top end of the upper column due to  $P_{Hi}$ . The  $P-\Delta$  effect due to the displacement of the vertical load  $P_v$  at the top end of the upper column is included. The analysis is conducted for the incremental values of  $P_{Hi}$ .

In the experiments, horizontal frictional forces are induced at the bearings, at the ends of the beams and at the top end of the upper column, due to the reactions from the axial load. The frictional forces increase the stiffness of the experimental lateral load versus displacement curve. For a precise prediction of this behaviour, it is decided to include the frictional forces in the computational model. First, the bearings are tested to determine their coefficients of friction. From the tests, it is found that the roller bearings at the beam ends have the coefficient of friction  $\mu_r$  equal to 0.11. The coefficient of friction for the bearing assembly at the top of the column,  $\mu_s$ , is 0.035. Next, a frictional force at a bearing is calculated by multiplying the vertical reaction at the bearing, corresponding to a load step, with the coefficient of friction. The value of each vertical reaction is calculated from statics due to  $P_v$  or  $P_H$  or both.

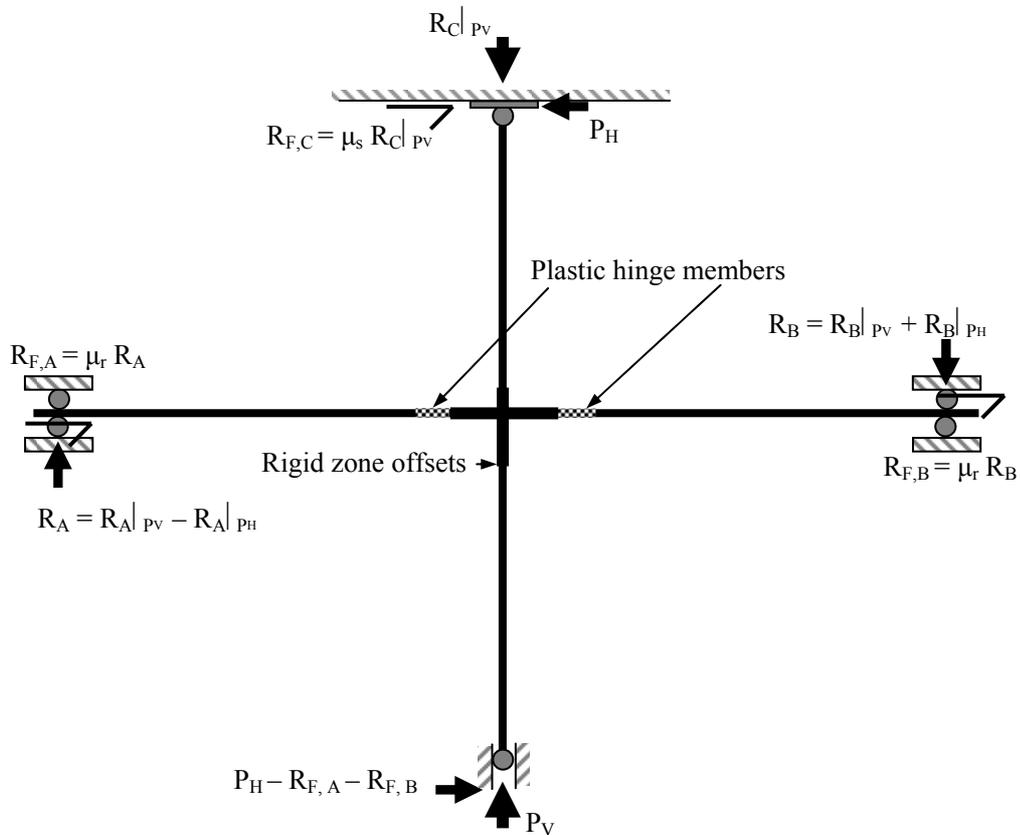


Fig. 20 Computational model for a sub-assembly specimen

From the incremental nonlinear analysis, the predicted lateral load versus displacement curve for the retrofitted specimen tested under the monotonic lateral loading is plotted in Figure 21. It can be observed that the predicted curve is close to the experimental results almost up to the peak. Near the peak, the experimental results deviate from the predicted curve due to the widening of the cracks in the jacket at the corners of the joint.

**GUIDELINES FOR PROFESSIONAL PRACTICE**

**1. Analysis of Retrofitted Sections**

A layered analysis can be adopted for a retrofitted section considering different strengths and behaviour of the existing concrete and the concrete in the jacket. A preloaded section can be analyzed by considering initial strain in the existing section. Neglecting slippage, the strain differences at the interface

of the two concrete should be maintained to satisfy the strain compatibility. A simplified analysis considering a uniform compressive strength (equal to that of the lower-grade concrete) throughout the section can be adopted for quick calculations.

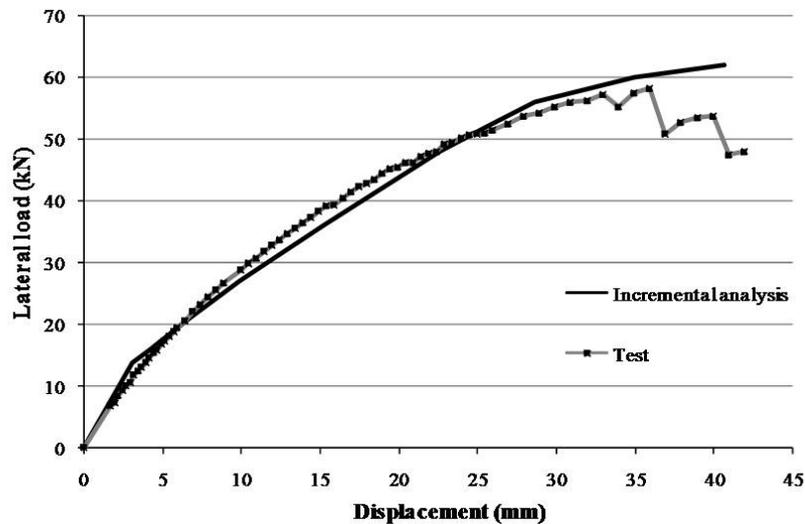


Fig. 21 Comparison of lateral load versus displacement curves for the retrofitted sub-assembly tested under monotonic loading

## 2. Design of Additional Bars for a Beam

- The required additional longitudinal bars for a beam are to be calculated based on the moment demand from the worst combination of gravity and seismic loads. However, the amount of additional reinforcement should be limited to ensure that the supporting column is still stronger than the beams.
- The additional stirrups and their spacing are to be calculated based on the shear demand from the capacity-based design expression and the requirement of confinement (BIS, 1993). This is to ensure the ductile detailing of the potential plastic-hinge regions.

## 3. Detailing

- In order to avoid any damage in the primary frame members made of poor concrete, the drilling of holes should be minimized in selecting a scheme of reinforcement for the concrete jacket. The additional longitudinal bars (that may be bundled, if required) can be placed near the corners of the beam and cranked around the joint with a splay of 1 in 10 or higher. This will avoid drilling holes through the joint. Although stirrups are provided along the length of the splay, at the spacing required for a potential plastic hinge, extra stirrups should be provided at the beginning of the splay (i.e., at the narrower section) to reduce the tendency of straightening of the bars and consequent breaking of the jacket concrete. A similar scheme may be adopted for the transverse beam, if retrofitted. The bars are to be tied with the added longitudinal bars in the column for the transfer of moment.
- The splaying of the bars and the jacket may not be architecturally acceptable for a column sufficiently wider than the beam. In such a case, drilling through the joint may be inevitable. However, this should be done with caution, since drilling reduces the shear capacity of the joint and may damage some existing bars in the column.
- The additional stirrups can be made of U-shaped bars to avoid drilling of the beams. The bars with end hooks can be hung and secured by binding wires to a holder bar on each side of the web of a beam. Alternately, they can be welded to the holder bars, provided weldable bars are used. The holder bars can be attached to the slab above the beam by J-hooks and nuts at a suitable interval.
- The additional stirrups can be closed (by using lapped U-bars and drilling through the slab) for an improved confinement, provided the existing concrete is of good quality and the drilling is executed carefully.
- If a building was not designed for the seismic forces, it is expected that the columns are inadequate in terms of flexural and shear capacities. The joints may also not have adequate shear capacity. The proposed detailing considers these factors along with the possible deficiency in the positive flexural

capacity of the beam. The columns should also be jacketed with additional reinforcement for maintaining a strong-column-weak-beam design. The joint can be confined by welding angle sections to the additional longitudinal bars in the columns. The details of column jacketing and strengthening of joints are illustrated in Kaliyaperumal and Sengupta (2009).

- For a beam wider than 300 mm, distributed longitudinal bars across the soffit should be provided. These bars are not continuous at the joint. However, they should be anchored to the added ties of the column with end hooks. For a beam with a large depth, additional side-face reinforcement can be provided as per the conventional design.
- A minimum cover should be provided as per IS 456 (BIS, 2000) to satisfy the fire safety and durability requirements.

#### **4. Construction**

- If possible, a beam should be subjected to reduced service loads before the casting of the jacket. The slab can be supported on the props adjacent to the beam.
- After chipping any plaster, the surface of the existing concrete should be roughened without damaging the concrete, especially in the case of poor quality of existing concrete. Hence, roughening by a motorised wire brush is preferred to hacking. To avoid the weakness of a cold joint, sand blasting the surface can provide more roughness by exposing the aggregates. Holes are drilled in the slab as per the spacing of the J-hooks (see Figure 22(a)).
- The surface should be thoroughly cleaned with a jet of water to remove the dust and wetted before jacketing.
- The new reinforcement cage can be fabricated at the floor beneath, lifted to the underside of the beam, and attached to the slab by J-hooks and nuts (see Figures 22(b) and 22(c)). The additional longitudinal bars and ties for the column are then placed. The longitudinal bars are continuous through the slab and welded with angle sections near the joint. They are tied with the additional longitudinal bars for the beam.
- Self-compacting concrete (SCC) or shotcrete can be used for the jacket. The flowability of SCC is important with regard to the filling up of the annular space between the existing concrete and the formwork. A shrinkage-compensating admixture needs to be added in the new concrete.
- The concrete jacket should be adequately cured before the beam is reloaded.

#### **SUMMARY AND CONCLUSIONS**

The present study has investigated the effect of jacketing on the positive flexural capacity and performance of beams near an interior joint. First, the testing of the reference and retrofitted simply supported beam specimens under a two-point bending has been documented. Second, the testing of the beam-column-joint sub-assembly specimens has been explained. Third, the prediction of the strength and behaviour of the retrofitted beam specimens has been illustrated. A layered analysis and a simplified analysis have been used for the predictions. The incremental nonlinear analysis for predicting the lateral load versus displacement behaviour of a sub-assembly has been highlighted. Finally, guidelines have been provided for the retrofitting of beams by concrete jacketing.

Following conclusions have been obtained from the present study:

- From the beam tests, it has been observed that the increase in strength and retention of ductility after concrete jacketing is as predicted by the analysis.
- The retrofitted specimens do not show a visible delamination between the existing concrete and the concrete in the jacket. The roughening of the surface of the existing concrete by motorised wire brush has been found to be satisfactory for the type of tests conducted.
- For the selected scheme of jacketing, the retrofitted sub-assembly tested under the monotonic loading has shown expected increase in the lateral strength. The ductility after retrofitting is found to be adequate.
- The degradations of the strength and stiffness of the retrofitted sub-assembly under the cyclic loading are gradual. Energy dissipation increases after the retrofitting. However, pinching in the hysteresis loops cannot be reduced.
- A layered analysis provides a good prediction of the strength and the moment versus rotation behaviour of a retrofitted section. The simplified analysis is also sufficiently accurate.

- The prediction of the lateral load versus displacement behaviour of a sub-assembly, by a conventional pushover analysis based on a bilinear (up to the peak) moment versus rotation curve for a lumped plastic hinge, is approximate, especially in the pre-yield region.
- The incremental nonlinear analysis with varying flexural stiffness for the hinge members (to model the spread plasticity), and incorporating friction of the bearings, has shown a substantially better prediction of the lateral load versus displacement behaviour of the retrofitted sub-assembly specimen as compared to the conventional pushover analysis.

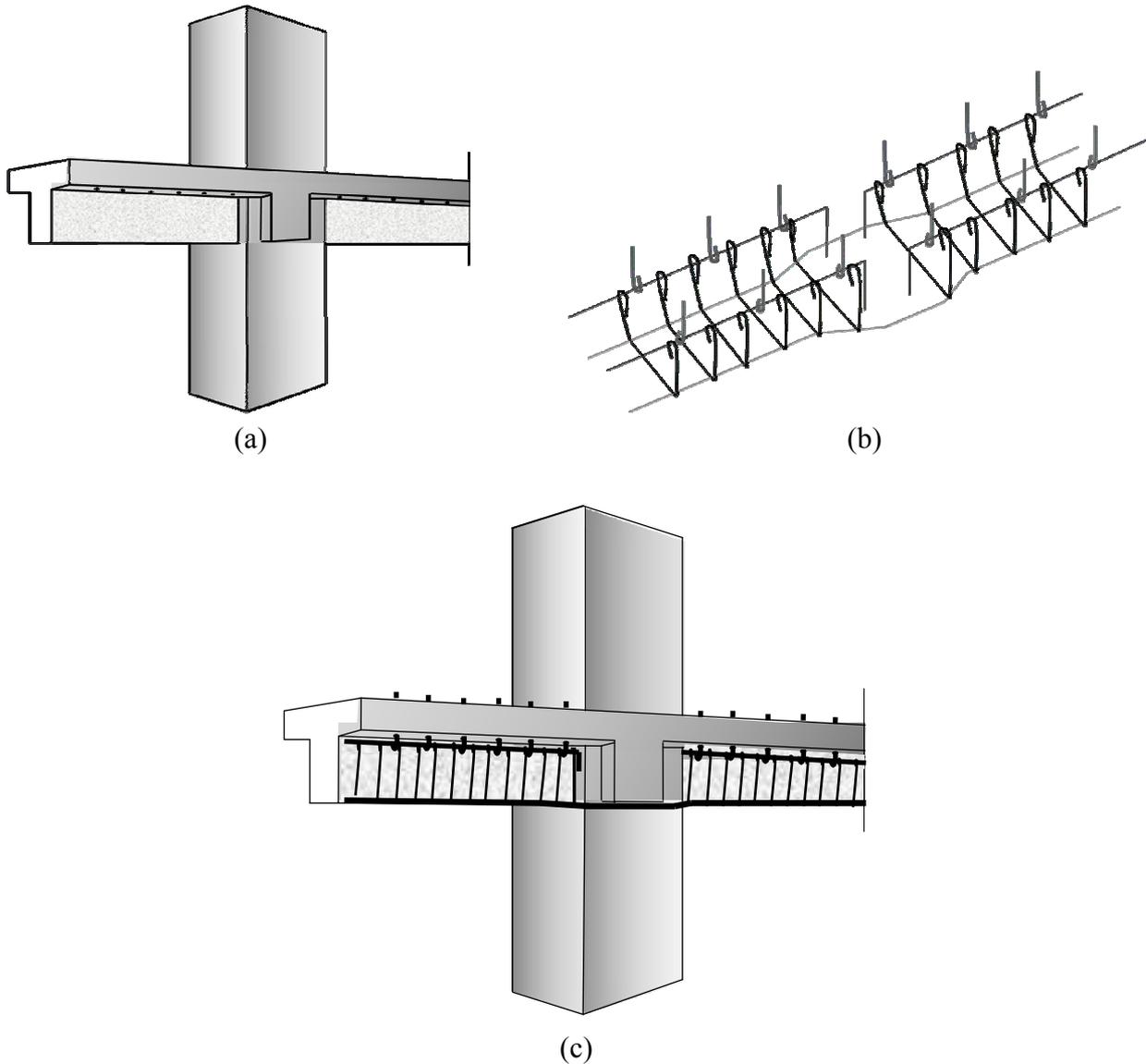


Fig. 22 Scheme of jacketing for enhancing positive flexural capacity of a beam near an interior joint (column bars not shown for clarity): (a) drilling of holes and surface preparation; (b) fabrication of reinforcement cage; (c) installation of reinforcement cage

The scheme of concrete jacketing selected in the present study needs to be qualified under a fast cyclic loading. Under this type of loading, the bond of the reinforcing bars deteriorates, leading to a drop in the strength. This study can be extended to the exterior beam-column joints by testing the corresponding sub-assembly specimens retrofitted with a suitable jacketing scheme. The strengthening of beams for shear also needs to be studied, especially on considering the high percentage of shear deficiencies of beams in the existing buildings. Three-dimensional frames with jacketed members can be tested under monotonic or cyclic lateral loads or base excitation applied by using a shake table.

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**REFERENCES**

1. Alcocer, S.M. and Jirsa, J.O. (1993). "Strength of Reinforced Concrete Frame Connections Rehabilitated by Jacketing", *ACI Structural Journal*, Vol. 90, No. 3, pp. 249–261.
2. Badari Narayanan, V.T. (2010). "Seismic Retrofit of Reinforced Concrete Beams in Buildings for Sagging Moment Using Concrete Jacket", Ph.D. Thesis, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai.
3. Bhedasgaonkar, B.V. and Wadekar, M.K. (1995). "Repairs of Beams and Slab with Excessive Deflection", *The Indian Concrete Journal*, Vol. 69, No. 1, pp. 47–50.
4. BIS (1993). "IS 13920: 1993—Indian Standard Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces—Code of Practice", Bureau of Indian Standards, New Delhi.
5. BIS (2000). "IS 456: 2000—Indian Standard Plain and Reinforced Concrete—Code of Practice (Fourth Revision)", Bureau of Indian Standards, New Delhi.
6. Chakrabarti, A., Menon, D. and Sengupta, A.K. (editors) (2008). "Handbook on Seismic Retrofit of Buildings", Alpha Science International Ltd., Oxford, U.K.
7. Cheong, H.K. and MacAlevey, N. (2000). "Experimental Behavior of Jacketed Reinforced Concrete Beams", *Journal of Structural Engineering*, ASCE, Vol. 126, No. 6, pp. 692–699.
8. Kaklauskas, G. and Ghaboussi, J. (2001), "Stress-Strain Relations for Cracked Tensile Concrete from RC Beam Tests", *Journal of Structural Engineering*, ASCE, Vol. 127, No. 1, pp. 64–73.
9. Kaliyaperumal, G. and Sengupta, A.K. (2009). "Seismic Retrofit of Columns in Buildings for Flexure Using Concrete Jacket", *ISET Journal of Earthquake Technology*, Vol. 46, No. 2, pp. 77–107.
10. Liew, S.C. and Cheong, H.K. (1991). "Flexural Behavior of Jacketed RC Beams", *Concrete International*, Vol. 13, No. 12, pp. 43–47.
11. Murty, C.V.R., Goel, R.K., Goyal, A., Jain, S.K., Sinha, R., Rai, D.C., Arlekar, J.N. and Metzger, R. (2002). "Reinforced Concrete Structures", *Earthquake Spectra*, Vol. 18, No. S1, pp. 149–185.
12. Sengupta, A.K., Satish Kumar, S.R., Badari Narayanan, V.T. and Gnanasekaran, K. (2006). "Facility for Testing Beam-Column Sub-assemblages", *Proceedings of the 13th Symposium on Earthquake Engineering*, Roorkee, pp. 925–933.
13. Shehata, I.A.E.M., Shehata, L.C.D., Santos, E.W.F. and Simões, M.L.F. (2009). "Strengthening of Reinforced Concrete Beams in Flexure by Partial Jacketing", *Materials and Structures*, Vol. 42, No. 4, pp. 495–504.