SEISMIC RETROFIT OF COLUMNS IN BUILDINGS FOR FLEXURE USING CONCRETE JACKET

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ABSTRACT

To prevent disaster in future earthquakes, the existing deficient buildings need to be retrofitted. One way of retrofitting the columns in reinforced concrete multistoreyed buildings is concrete jacketing. The present study has investigated the effect of jacketing on the flexural strength and performance of columns. First, slant shear tests were conducted to study the interface between the old and new concrete. Second, column specimens were tested to study the strength. Third, beam-column-joint sub-assemblage specimens were tested to study the ductility (or energy absorption) and energy dissipation. Analytical investigations were carried out to predict the experimental results. A lamellar approach and a simplified method of analysis were used for the prediction of the axial load versus moment interaction curves and moment versus curvature curves for the retrofitted columns. An incremental nonlinear analysis was adopted to predict the lateral load versus displacement behaviour for a retrofitted sub-assemblage specimen. Guidelines for the retrofitting of columns by concrete jacketing are provided.

KEYWORDS: Beam-Column-Joint Sub-assemblage, Column, Concrete Jacketing, Lamellar Analysis, Retrofit

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. Failure of columns can lead to the failure of a storey and the building. The columns in a typical multistoreyed RC building in India, especially with an open-ground storey (i.e., a ground storey without any infill walls for vehicle parking), are found to be deficient with respect to their flexural and shear strengths as compared to the corresponding demands. Under moderate to severe earthquakes, an undesirable column side-sway can lead to a soft-storey collapse mechanism.

The Department of Civil Engineering at Indian Institute of Technology Madras and the Structural Engineering Research Centre, Chennai undertook a project to study the vulnerability of typical multistoreyed RC buildings in India. The project was funded by Department of Science and Technology, Government of India. From the analysis of 18 residential buildings, it was found that on average 22% and 39% of the lower storey columns are deficient in flexure for the Zone III and Zone V buildings, respectively (Kamat and Sengupta, 2006). For the two zones, the horizontal components of peak ground acceleration are $0.16g$ and $0.36g$, respectively, where g is the acceleration due to gravity. For deficiency in shear, the corresponding values are 84% and 54%, respectively. It was observed that for some buildings, all columns in the lowest storeys are deficient in shear. Since shear failure in columns in a storey can lead to the pancake type collapse of a building, many multistoreyed buildings in India are highly vulnerable.

The deficiencies in the buildings, which were damaged or collapsed during the Bhuj earthquake, were reported by Murty et al. (2002). A few other observations were made in the project mentioned earlier (IITM-SERC, 2005).

Apart from an inadequate number of continuous frames in both directions, improper layout and orientation of columns, common deficiencies specific to the columns in RC buildings include:

- Inadequate and disproportionate dimensions of the column section, leading to inadequate flexural and shear capacities in the columns of open ground storey. The practice of providing 230 mm wide columns in the lower storeys may result in significant inadequacy in strength.
- Inadequate spacing of ties at the potential plastic hinge regions, leading to lack of ductility.
- Inadequate flexural capacity due to weak construction joints, and poor detailing of ties, such as warped ties (i.e., the ties which are not in a single plane after bending the bar manually), and lack of adequate end hooks.
- Location of splice for the longitudinal bars just above the floor level, and inadequate splice length as compared to the requirement based on tension in the bars.
- Short captive columns created in the presence of partial-height infill walls and openings in the walls.
- Lower concrete strength than the design strength, and concrete of poor quality with inadequate compaction at the top of the columns.

To prevent disaster in future earthquakes, the existing deficient buildings need to be retrofitted. 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. Retrofit strategies may be broadly classified as local and global strategies (see Figure 1). 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 retrofit some columns. The latter can be considered as a local retrofit strategy.

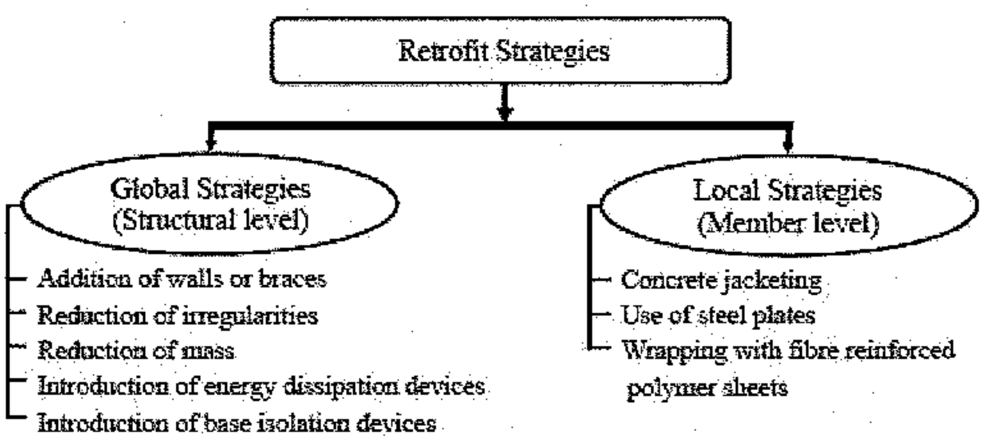


Fig. 1 Types of retrofit strategies

One way of retrofitting the columns is by concrete jacketing. Concrete jacketing involves placing an additional layer of concrete covering the existing column, together with additional longitudinal bars and ties to enhance the flexural and/or shear capacities. The retrofitting of columns by concrete jacketing is not sufficiently documented. In a conventional analysis of a jacketed column, strength is determined based on an interaction diagram for the composite section or for some equivalent section. The present paper reports an investigation of the strength of jacketed columns and the performance of jacketed columns in beam-column-joint sub-assemblages. Only the flexural deficiency of such columns is addressed in this study.

In the conventional method, a column is analysed or designed by using the interaction charts of axial load and bending moment. This is a strength-based approach. However, for analyzing and designing for the seismic forces, a performance-based approach is preferred. In this approach, not only the strength of a structural component is evaluated, but also the load versus deformation behaviour is quantified. Next, testing a beam or column specimen under a static monotonically increasing load does not demonstrate the behaviour of members near the beam-to-column joints under seismic loads, which are dynamic and cyclic in nature. The degradation of strength and stiffness of the members and deterioration of bond between the steel reinforcing bars (or rebars) and concrete, especially near the joints, cannot be simulated by the monotonic loading of individual specimens. Hence, it is necessary to test the beam-column-joint sub-assemblages under at least a quasi-static or pseudo-dynamic loading in order to study the effect of retrofit.

LITERATURE REVIEW

The 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 column.
- The compatibility of deformation between old and new concrete, and the durability are better as compared to a new material on a different substrate.
- Availability of personnel skilled in concrete construction.
- Analysis of retrofitted sections follows the principles of analysis of RC sections.

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

- increase in the size of the column and reduction in floor space,
- anchoring of bars for flexural strength involves drilling of holes in the slabs and footings,
- manufacturing of sufficiently workable concrete for the jacket, and
- possibility of disruption to the users of the building.

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

Aguilar et al. (1989) studied 114 buildings which were retrofitted after the Mexico earthquake in 1985. It was concluded that the most commonly used retrofit strategy in the buildings less than 12 storeys was that of concrete jacketing of columns.

Bett et al. (1988) have shown that columns strengthened by jacketing, both with and without supplementary cross-ties, were much stiffer and stronger than the original, unstrengthened columns. Saatcioglu and Ozcebe (1989) pointed out that a cross-tie with 90° hook at one corner and with extension of 10 times the bar diameter performs as satisfactorily as that with 135° hook at both ends and with similar extension. Bousias and Fardis (2003) proved that concrete jacketing is very effective in removing the adverse effect of lap splicing on flexural capacity, even for very short lap lengths.

Alcocer (1993) (also Alcocer and Jirsa, 1993) tested beam-column-slab specimens under bidirectional cyclic loading to study the effect of jacketing on the shear strength of joints. Different layouts of the additional longitudinal bars of the columns were studied. It was concluded that although distributed bars are preferred for good bond behaviour, bundling of bars near the corners of the column were effective in increasing the strength, stiffness and energy dissipation characteristics of the specimens. Ersoy et al. (1993) concluded that a retrofitted column behaved better when the jacket was constructed after unloading the column.

Valluvan et al. (1993) tested column specimens with lap splicing of the longitudinal bars. From the tests it was concluded that removing the concrete cover for adding the new ties is not an effective method for strengthening the splice location because it results in micro-cracking of the concrete core. External reinforcement (i.e., steel element or tie) around the splice region significantly improves the confinement of the concrete and the strength of the splice. Steel dowels were inserted at the face of the original columns for better transfer of shear at the interface of the old concrete and the jacket.

Rodriguez and Park (1994) tested jacketed columns with stub beams. The additional longitudinal bars in a specimen were placed in bundles near the corners of the column and passed through drilled holes in the slab. Additional ties at the joint were inserted by drilling the stub beams. Significant improvements of the lateral stiffness, strength and ductility of the specimens were observed.

Vandoros and Dritsos (2006) designed and tested a typical ground storey column of a framed building under displacement-controlled cyclic loading. Self-compacting concrete was used in the jacket. The longitudinal bars of the original column were connected to the additional longitudinal bars in the jacket with steel inserts. It was found that an almost monolithic behaviour could be achieved, when the jacket was constructed with dowels but without any treatment of the old surface. Significant increases in strength and stiffness were observed.

Regarding the bonding between old and new concrete, Stoppenhagen et al. (1995) roughened the column surface with an electric concrete hammer to achieve a monolithic behaviour of the entire concrete. Abu-Tair et al. (1996) adopted the slant shear test and modified modulus of rupture test to study the effect of surface treatment on the bond strength at the interface, under monotonic and cyclic loadings. It was concluded that both tests are effective for studying the bond strength.

Austin et al. (1999) compared the bond strength through tensile pull-out tests. The effect of surface preparation and the mismatch of the moduli of the repair material and substrate were studied. A failure envelope was presented based on the Mohr-Coulomb theory and Griffith fracture criterion.

Tests for bond and the effects of bond coats, surface preparation and ageing of the base concrete were conducted by Climaco and Regan (2001). The test results were analyzed by using the Coulomb criterion. When the base surface is dry, good bond can be achieved by casting the repair material against the old concrete without any bonding agent. Julio et al. (2003) investigated the effects of surface preparation, bonding agents and steel connectors. It was concluded that the monolithic behaviour of a jacketed member can be achieved by using bonding agents or steel connectors, without increasing the surface roughness.

Beushausen and Alexander (2008) tested the following types of surface preparation:

- sand blasted,
- plain, without any intentional roughening, and
- notched surface.

From the tests it was found that the notched surface showed higher bond strength than the sand blasted and the plain interfaces, even at 26 months after casting of the new concrete.

SLANT SHEAR TESTS

For an effective concrete jacket, the use of dowels involves closely spaced drilling of the existing concrete. If the latter is of low strength or poor quality, the drilling may worsen the existing member. Also, the drilling may intercept the existing bars. Hence, to minimize drilling, the use of dowels is precluded and only preparation of the surface of the existing member is investigated in the present study of retrofit for flexure. Slant shear tests were carried out in the present investigation to check the interface bond between old and new concrete.

1. Test Setup

The tests were conducted with the square prisms made of new concrete cast against the sloping faces of the old concrete, as shown in Figure 2. The procedure as per BS 6319 (BSI, 1984) was followed. The specimen size was 150×75×75 mm. The old concrete was cured for 28 days before casting of the new concrete. For different specimens, the faces of old concrete were either plain without any intentional roughening, or roughened by motorized wire brush, or were hacked by chisel. For each of these types of surfaces there were two varieties: one without any application of the bonding agent, and another with the smearing of bonding agent.

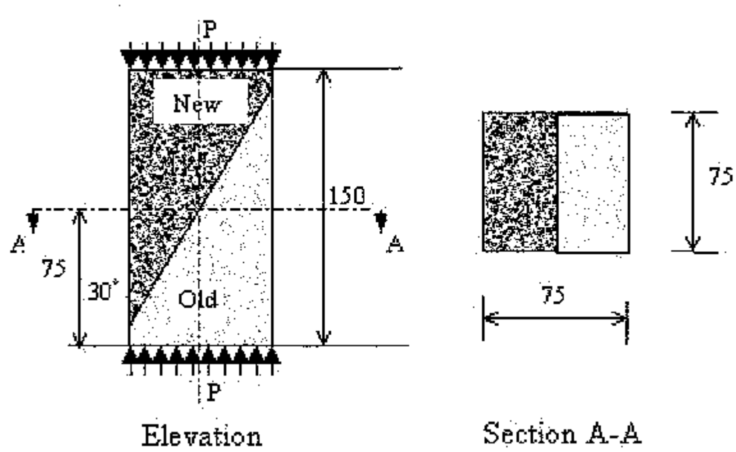


Fig. 2 Schematic diagram for the slant shear test (all dimensions are in mm)

The prisms were tested under compression after an additional 28 days of curing. The average values of the individual cube strengths of the old and new concrete were 32 and 44 N/mm^2, respectively.

2. Test Results

The typical failure pattern of a specimen with roughened surface is shown in Figure 3. The failure loads are given in Table 1. Figure 4 shows the corresponding values of average failure stress. From the results, it was found that the specimens with roughened surface of the old concrete and without any bonding agent failed at higher loads compared to the other specimens. Thus, it was decided that for the retrofitted column and sub-assemblage specimens, the faces of the old concrete would be roughened and bonding agent would not be used.

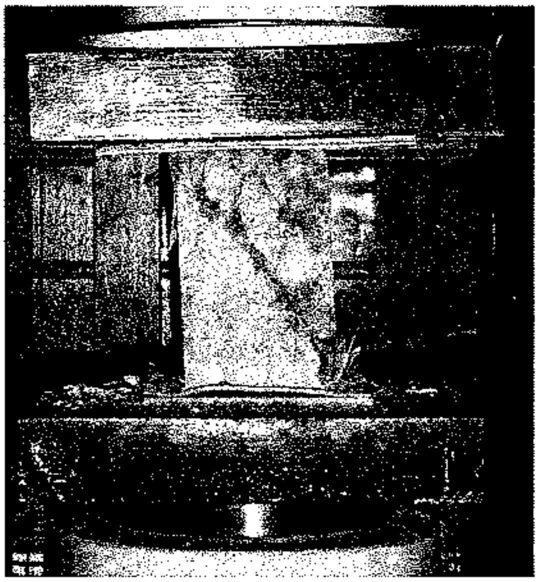


Fig. 3 Failure pattern of a specimen with roughened surface

Table 1: Failure Loads (in kN) for Different Specimens of Slant Shear Tests

Surface Preparation	A1	A2	A3	B1	B2	B3	C1	C2	C3
	Without Chemical			With Chemical 1			With Chemical 2		
Plain	No test was performed			25	35	30	10	05	–
Roughened	110	135	170	15	55	50	10	05	10
Hacked	140	90	90	35	35	20	05	10	–
Specimens C3 with the plain and hacked surface preparations were defective.									

TESTS OF COLUMNS

It is necessary to predict the strength of a retrofitted column in order to estimate the effect of retrofit. In this study, first the columns were tested to investigate the increase in axial load capacity and flexural strength due to the concrete jacketing. To develop interaction curves for the reference (with suffix O) and retrofitted (with suffix R) specimens, nine specimens were tested for each case. For each case, three specimens were tested under each of pure compression (PC), eccentric compression (EC), and pure uniaxial bending (PB).

1. Test Setup

Column specimens were tested under pure compression and eccentric compression by using a column testing machine. The specimens under pure bending were tested using a frame which is described later. The elevations of the test setups are shown in Figure 5. For a test under eccentric compression, the compressive load P was applied at an eccentricity e about one axis. The values of eccentricity were selected corresponding to the theoretical balanced failure points of the interaction curves and were rounded off to suitable integral values. Rocker bearings were placed at the top and bottom of the specimens to develop the pinned end conditions.

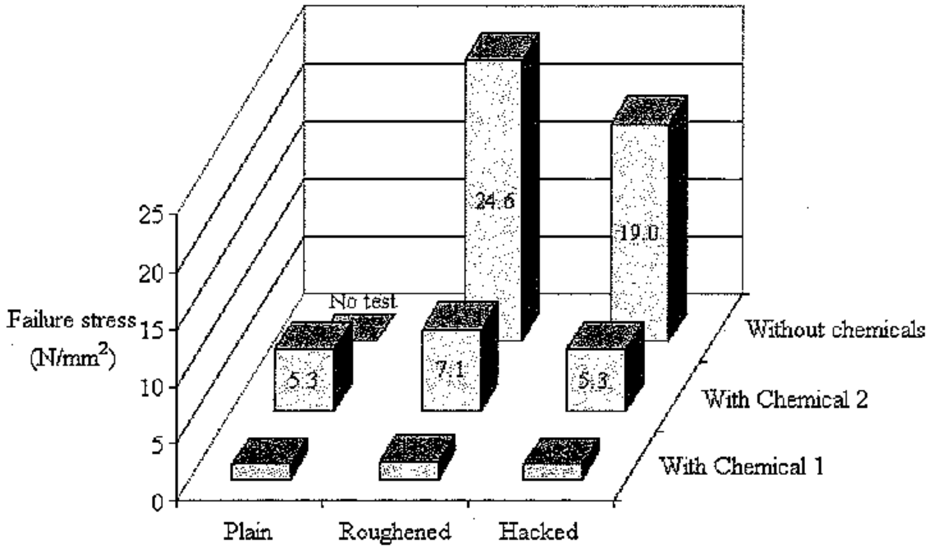


Fig. 4 Average failure stress for the specimens of slant shear tests

A load cell was used to record the applied load. Linear variable differential transducers (LVDTs) were used to measure the deformations over certain gage lengths. The average strains and curvatures were calculated from the deformation readings. A data acquisition system was used to record the load cell and LVDT readings.

2. Specimen Details

2.1 Reference Specimens

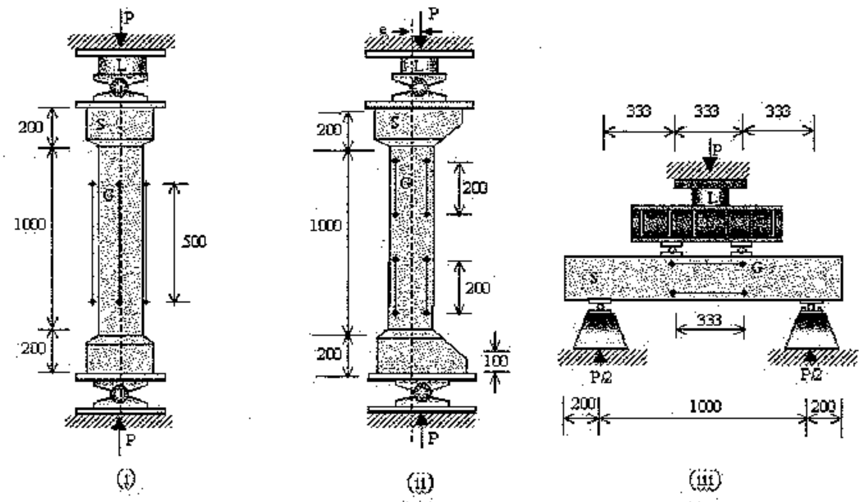
A reference specimen simulates the as-built column. The reinforcement details of the specimens are shown in Figure 6. To simulate the conventional practice, the end hooks of the ties of the reference specimens were provided with 90° bends instead of the 135° bends. For the specimens tested under pure compression, the ends were flared symmetrically to a cross-section of 200×200 mm to avoid stress concentration and subsequently local failure. For the specimens tested under eccentric compression, the ends were flared asymmetrically to a cross-section of 300×200 mm. For the reference specimens, a concrete mix of 1:2.35:4.7 (cement: fine aggregate: coarse aggregate) with water-to-cement ratio of 0.5 was used. Table 2 provides the material properties for the column specimens.

2.2 Retrofitted Specimens

For a retrofitted specimen, the cross-section, reinforcement detailing and concrete mix of the inner portion were similar to those of the corresponding reference specimen. The reinforcement details for the jackets are shown in Figure 7. For the specimens tested under pure compression, the cross-section of the flared ends was 300×300 mm. For the specimens tested under eccentric compression, the cross-section of the flared ends was 500×300 mm.

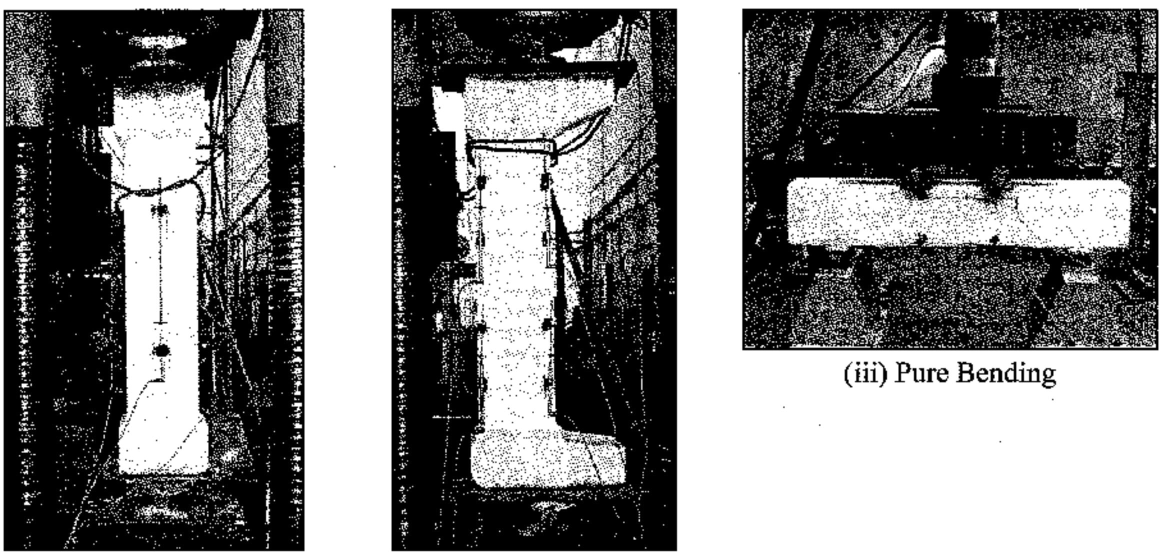
The jackets of the retrofitted specimens were made of self-compacting concrete (SCC). After several trials, a design mix of the SCC was formulated and flowability was tested by the slump flow test. Segregation of the concrete was not observed in the test. Photographs of the fabricated reinforcement for the retrofitted specimens are shown in Figure 8.

Spacing of the ties in the jacket was kept close (but limited to the minimum spacing) in order to increase the confinement of the concrete of the inner portion. The reference specimens and the inner portion of the retrofitted specimens were cast horizontally. Since the height of the specimens was 1400 mm (including the flares), segregation of the concrete was not expected even if cast vertically. The jackets of the specimens tested under pure compression and eccentric compression were cast vertically to simulate the actual method of construction. The jackets of the specimens tested under pure bending were cast horizontally in order to use an adjustable formwork.



B = Spreader beam, e = eccentricity, L = Load cell, G = LVDT gage, P = Applied load, S = Specimen
e = 50 mm for reference specimens and 100 mm for retrofitted specimens.

(a)



(iii) Pure Bending

(i) Pure Compression (ii) Eccentric Compression

(b)

Fig. 5 Setups for testing column specimens (all dimensions are in mm): (a) schematic diagrams, (b) photographs of the retrofitted specimens

3. Test Results

3.1 Reference Specimens

The specimens tested under pure compression failed due to lateral buckling of the longitudinal bars between the ties followed by the crushing of concrete (see Figure 9). This was expected as the tie spacing was large compared to the size of the longitudinal bars. The specimens tested under eccentric compression showed substantial cracking on the tension side and subsequently lateral buckling of the longitudinal bars between the ties and crushing of the concrete on the compression side. The lateral ties did not open under the static load. For the specimens tested under pure bending, the initiation of yielding of the longitudinal bars on the tension side was followed by the crushing of concrete on the compression side. The failure loads for the column specimens are given in Table 3. The loads were subsequently normalized to have non-dimensional ratios for plotting in a common graph.

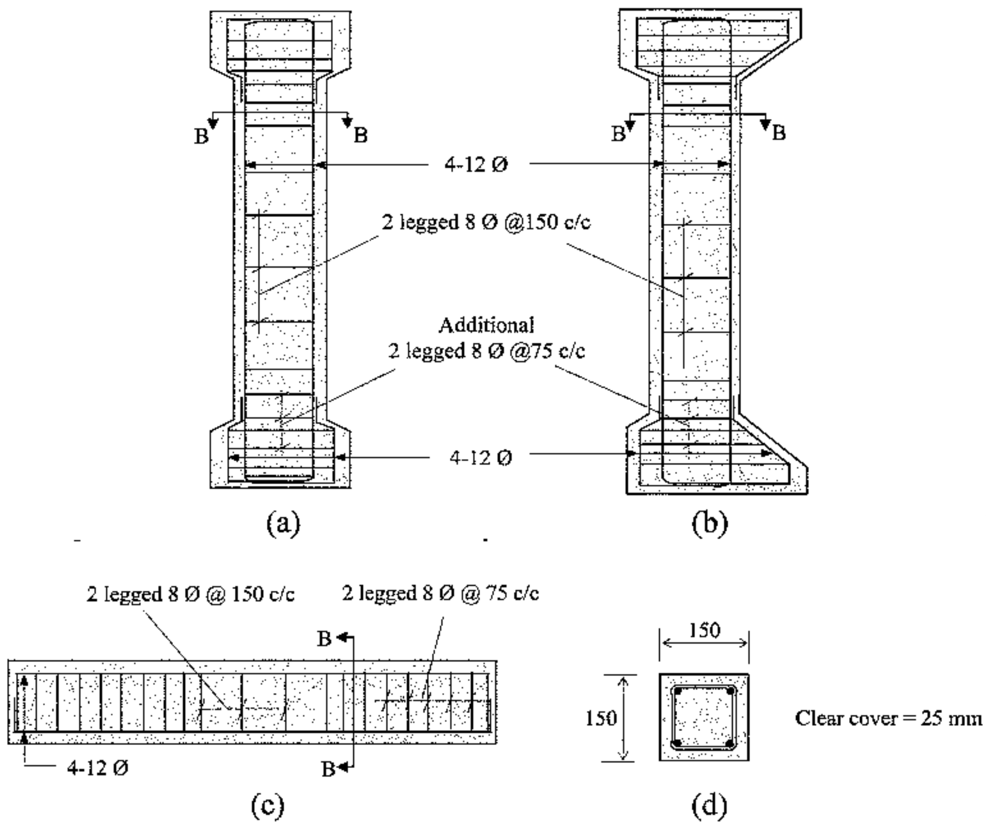


Fig. 6 Reinforcement details of the reference specimens for column tests (all dimensions are in mm): (a) pure compression, (b) eccentric compression, (c) pure bending, (d) Section B-B

Table 2: Material Properties for Column Specimens

Series	Reference		Retrofitted*			Reference and Retrofitted	
	Specimen Designation	f_{cm}	Specimen Designation	f_{cmE}	f_{cmJ}	f_{yl}	f_{yt}
Pure Compression (PC)	PCO 1	23	PCR 1	24	31	413	480
	PCO 2	31	PCR 2	24	43		
	PCO 3	22	PCR 3	24	24		
Eccentric Compression (EC)	ECO 1	23	ECR 1	33	20		
	ECO 2	31	ECR 2	45	21		
	ECO3	22	ECR 3	38	19		
Pure Bending (PB)	PBO 1	22	PBR 1	24	31		
	PBO 2	23	PBR 2	24	43		
	PBO 3	40	PBR 3	24	24		

f_{cm} = mean cube strength of concrete (in MPa), f_{cmE} = mean cube strength of existing concrete (in MPa), f_{cmJ} = mean cube strength of jacket concrete (in MPa), f_{yl} = yield strength of longitudinal bars (in MPa), f_{yt} = yield strength of transverse bars (in MPa), modulus of elasticity for steel = 2.02×10^5 N/mm^2

*In a few specimens, the compressive strength of concrete in the jacket came out to be lower than that for the existing concrete because of the different periods of curing and trial mixes of the self-compacting concrete.

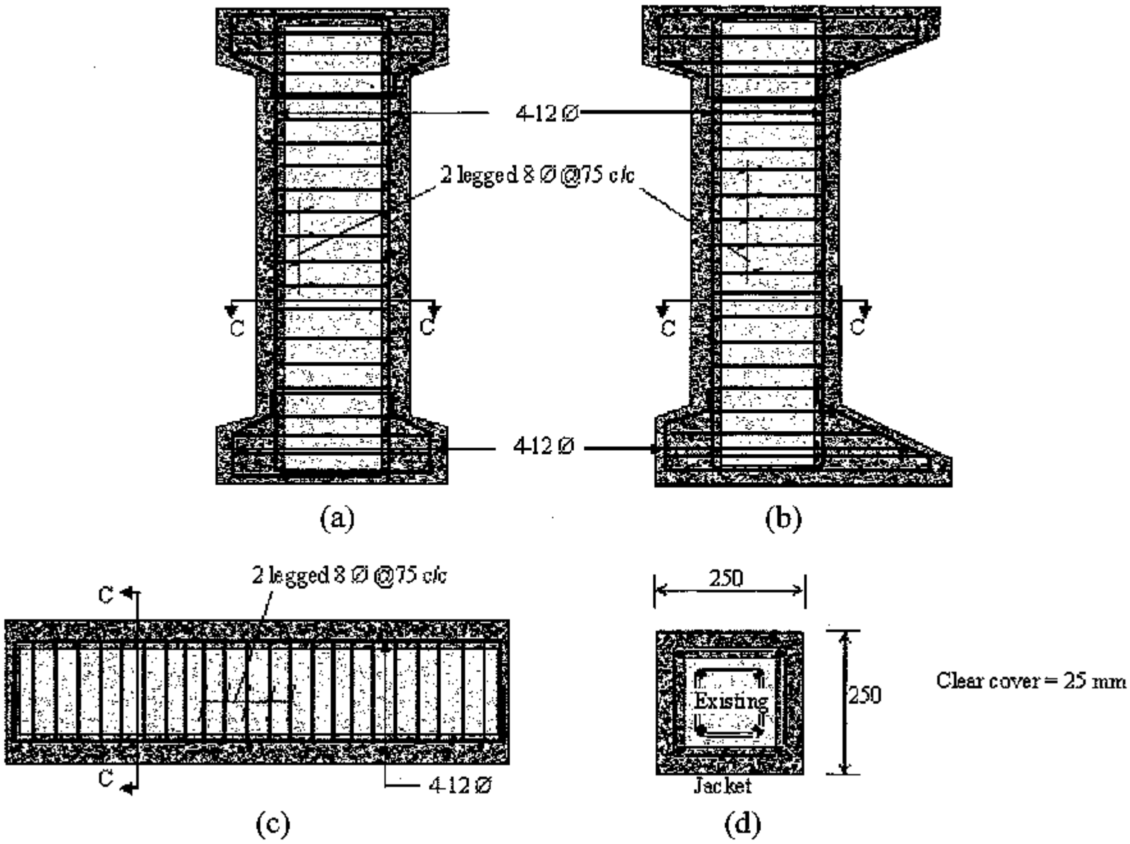


Fig. 7 Reinforcement details of the retrofitted specimens for column tests (all dimensions are in mm): (a) pure compression, (b) eccentric compression, (c) pure bending, (d) Section C-C (these bars are in addition to those in the original cross-sections, which are same as in the reference specimens; see Figure 6)

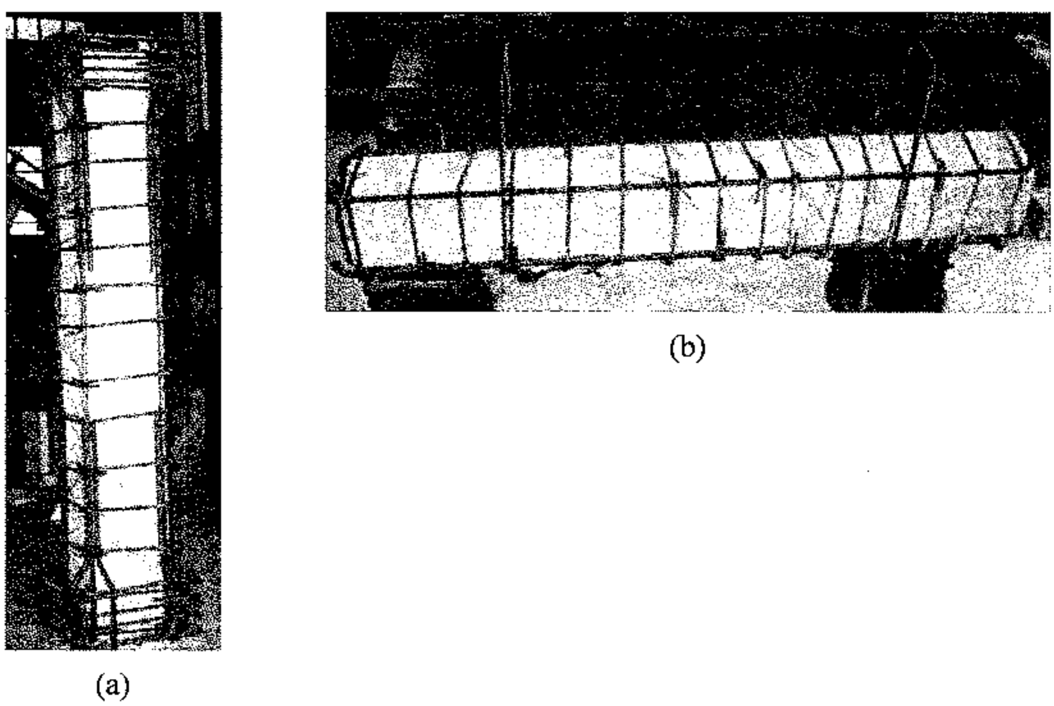
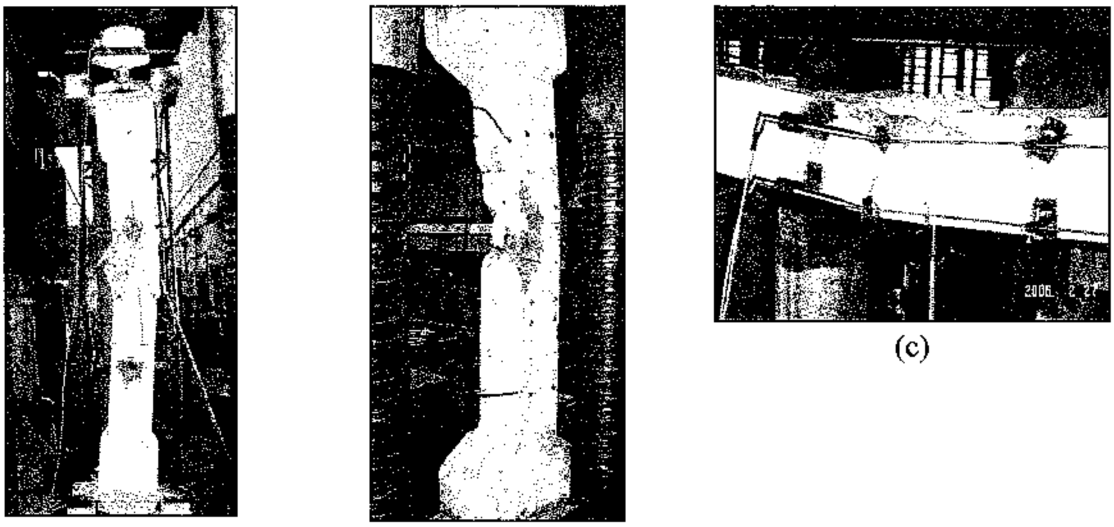


Fig. 8 Photographs of fabricated reinforcements of the retrofitted specimens for column tests: (a) pure compression, (b) pure bending



(a) (b) (c)

Fig. 9 Photographs of reference specimens after (typical) testing: (a) pure compression, (b) eccentric compression, (c) pure bending

Table 3: Failure Loads for Column Specimens

Type of Specimen	Specimen Designation	Axial Load P_{UR} (kN)	Moment M_{UR} (kN-m)	$\frac{P_{UR}}{f_{cm}BD}$	$\frac{M_{UR}}{f_{cm}BD^2}$
Reference	PCO 1	646	0	1.25	0
	PCO 2	720	0	1.03	0
	PCO 3	560	0	1.13	0
	ECO 1	250	12.5	0.48	0.16
	ECO 2	260	13.0	0.37	0.12
	ECO 3	250	12.5	0.51	0.17
	PBO 1	0	7.0	0	0.10
	PBO 2	0	13.3	0	0.17
	PBO 3	0	8.6	0	0.07
Retrofitted	PCR 1	1350*	0	0.90	0
	PCR 2	2150	0	1.43	0
	PCR 3	1565	0	1.04	0
	ECR 1	547	54.7	0.27	0.11
	ECR 2	506*	50.6	0.18	0.07
	ECR 3	573	57.3	0.24	0.10
	PBR 1	0	37.5	0	0.10
	PBR 2	0	36.5	0	0.10
	PBR 3	0	38.2	0	0.10

f_{cm} = mean cube strength of the concrete for a reference specimen, $f_{cm} = f_{cmE}$ = mean cube strength of the existing concrete for a retrofitted specimen, B = width of the column, D = depth of the column

*The specimen had local failure.

3.2 Retrofitted Specimens

The specimens tested under pure compression failed by crushing of the concrete of both jacket and core (see Figure 10). There was no visible delamination of the jacket. Also, light tapping by a hammer did not reveal any air gap or cause the jacket concrete to chip off. For the specimens tested under eccentric compression, there was no visible difference in the crushing of the old and new concrete. Specimens PCR 1 and ECR 2 showed premature local crushing of the concrete near the flares. For the specimens tested under pure bending, along with yielding of the bars and crushing of concrete under compression, portions of the jacket between the ties delaminated on the tension side.

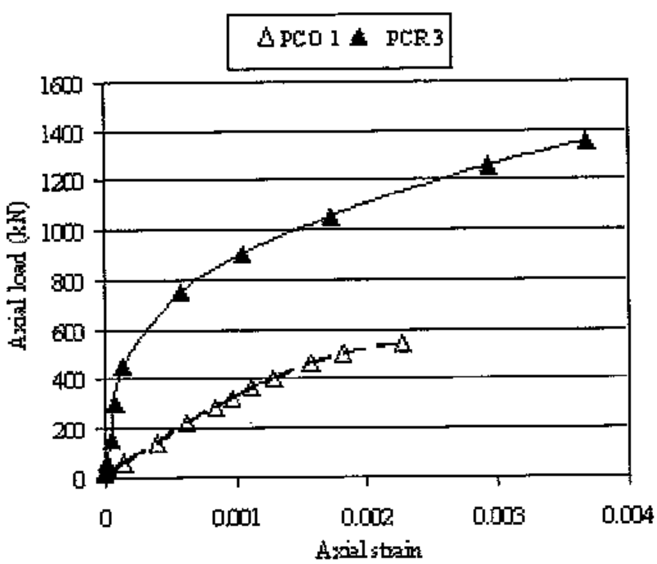
(a)

(b)

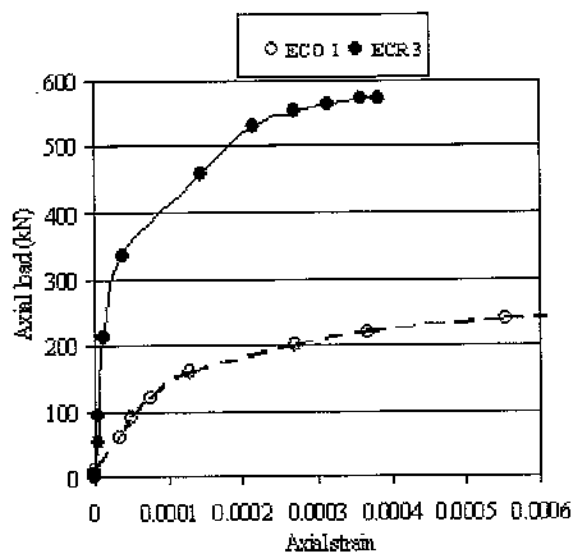
(c)

Fig. 10 Photographs of retrofitted specimens after (typical) testing: (a) pure compression, (b) eccentric compression, (c) pure bending

The axial load versus strain curves for typical reference and retrofitted specimens under pure compression and eccentric compression are compared in Figure 11. The increase in axial load capacities of the retrofitted specimens can be observed in this figure. The moment versus curvature curves for typical reference and retrofitted specimens under eccentric compression and pure bending are compared in Figure 12. These plots show an increase in the moment capacities of the retrofitted specimens with respect to the reference specimens.



(a)



(b)

Fig. 11 Axial load versus strain curves for reference (PCO 1 and ECO 1) and retrofitted (PCR 3 and ECR 3) specimens under (a) pure compression and (b) eccentric compression

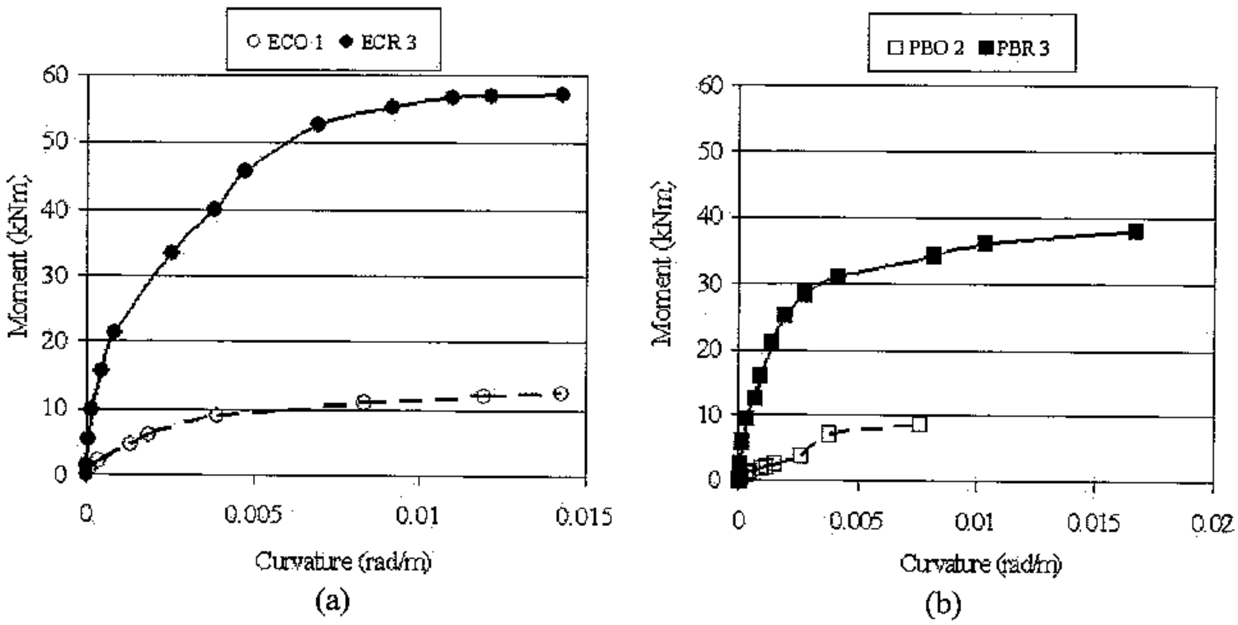


Fig. 12 Moment versus curvature curves for reference (ECO 1 and PBO 2) and retrofitted (ECR 3 and PBR 3) specimens under (a) eccentric compression and (b) pure bending

3.3 Interaction Plots

The failure load for each specimen is plotted in Figure 13. In the interaction plot, the vertical and horizontal axes represent the axial load and moment capacities, respectively. It may be observed that there was a substantial increase in the strength after the retrofitting of the sections.

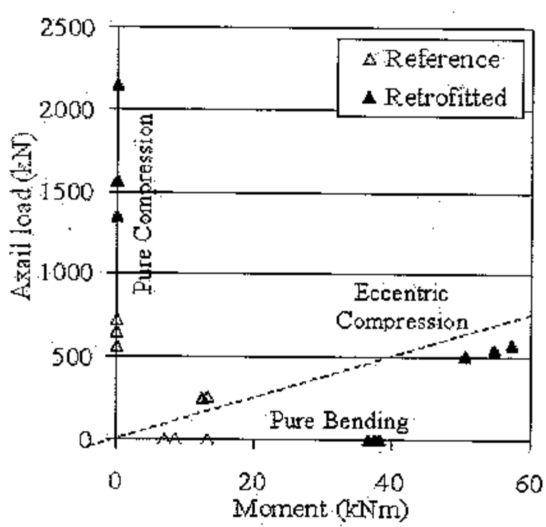


Fig. 13 Interaction plot of failure loads

TESTS OF BEAM-COLUMN-JOINT SUB-ASSEMBLAGES

To have an appropriate specimen for studying the effect of retrofitting, it is necessary to understand the actions of external loads on the components of a building. In a multistoreyed frame under lateral loading, the points of contraflexure lie approximately at the centres of the beams and columns (see Figure 14). The simulation of this condition for an interior component can be done by testing a beam-column-joint sub-assemblage under a lateral (i.e., horizontal) load P_H as shown in Figure 14(c). To consider the gravity load in the column, a simultaneous vertical load P_V needs to be applied. The bending moment diagram for the sub-assemblage is shown in Figure 14(d). The additional moments due to the lateral displacement of the applied vertical load (i.e., due to the P-Δ effect) are not shown. The total length and the total height of the sub-assemblage are represented as L and H, respectively.

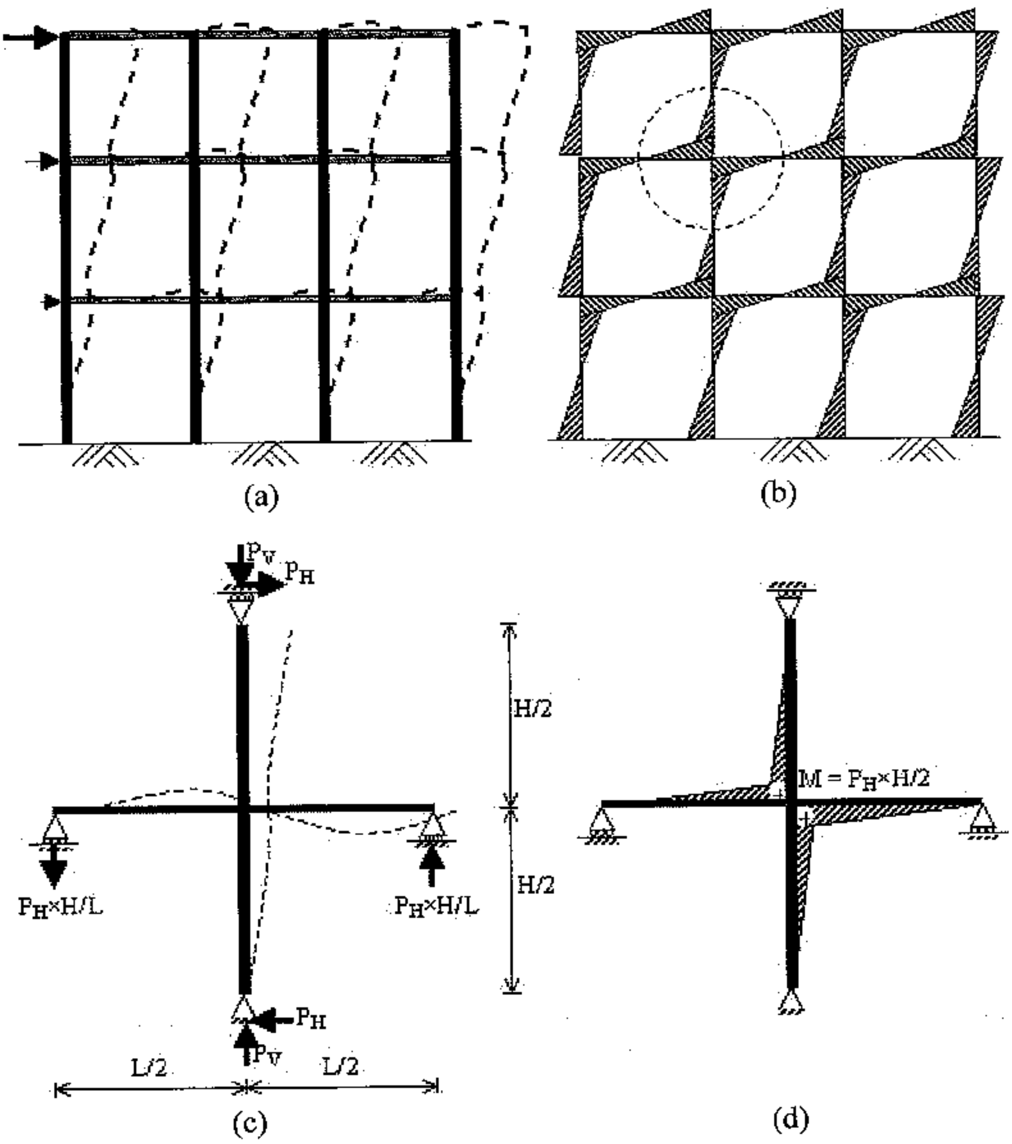
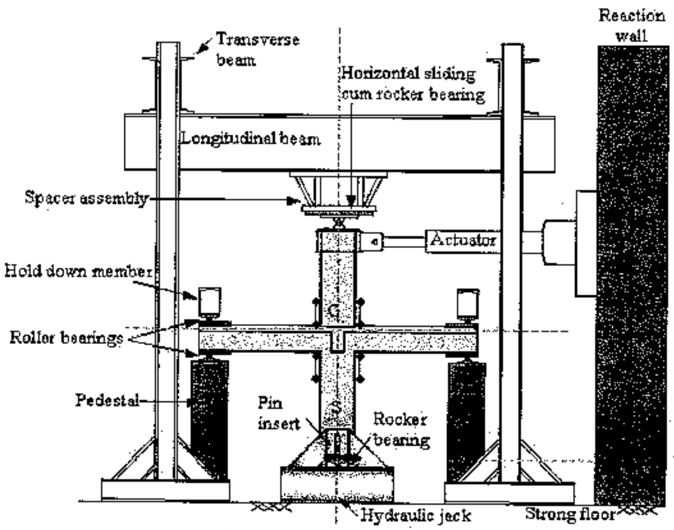


Fig. 14 Loading and bending moment diagrams for a frame and a beam-column-joint sub-assemblage: (a) moment-resisting frame under lateral load, (b) bending moment diagram for frame, (c) sub-assemblage, (d) bending moment diagram for sub-assemblage

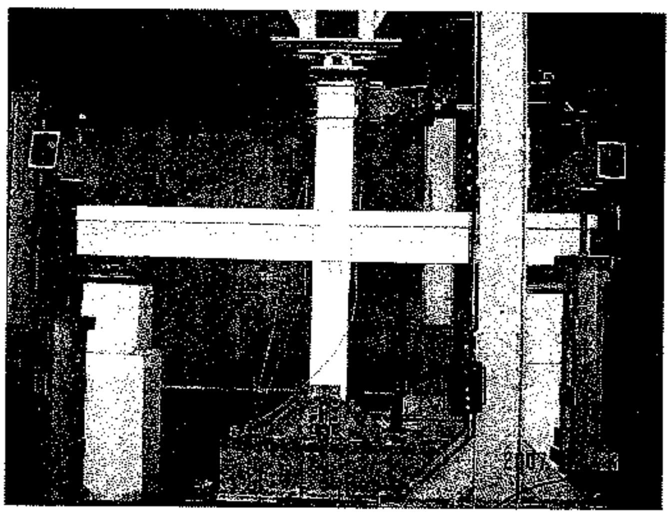
1. Test Facility

The sub-assemblage specimens were tested by using a recently developed test facility. The facility consisted of a reaction wall, a strong floor, and a test frame. The frame was designed and fabricated under the present study. The frame was also used to test the column specimens under pure bending as mentioned in the previous section. The details of the setup for testing the sub-assemblage specimens are shown in Figure 15. The top end of the upper column was attached to the frame through a spacer assembly and a horizontal sliding-cum-rocker bearing. This allowed the top end to rotate and translate horizontally. The lateral load was applied at this end by a servo-controlled actuator fitted to the reaction wall.

A rocker bearing was kept at the bottom end of the column. A vertical load of 130 kN was applied on the column, which approximates the axial load corresponding to the balanced failure of the reference column section. The vertical load was applied at the bottom end by a hydraulic jack. An assembly was provided at the bottom end to place the jack. It had a vertical slot through which the pin insert at the bottom end of the column could slide. This allowed the bottom end to rotate and to translate vertically. The ends of the beams were supported on pedestals. By providing roller-cum-rocker bearings, these ends were allowed to translate horizontally and to rotate. The hold-down steel members prevented any uplift of the ends of the beams. The details of the test facility are provided in Gnanasekaran (2009).



(a)



(b)

Fig. 15 Setup for testing beam-column-joint sub-assemblages: (a) schematic diagram, (b) photograph of retrofitted specimen

To study the lateral load versus displacement behaviour, two reference and two retrofitted specimens were tested. For each type, one specimen was tested under monotonic lateral load and the other under cyclic lateral load. The displacement-controlled cyclic load history is shown in Figure 16. At each displacement level, three cycles of loading were applied. The increment Δ_y in the displacement levels was taken same as the theoretical displacement corresponding to the yielding of the outer column bars.

2. Specimen Details

The objective of the present study is to investigate the effect of jacketing on the flexural strength of columns in an interior frame. Hence, failures of the beams, and shear failures of the columns or the joints were deliberately avoided. The total length and total height of each specimen were 2.5 and 3.0 m, respectively. Stub beams in the transverse direction and slab over the beams were provided to create obstructions in placing the additional longitudinal bars in the column jacket, like in an interior frame of an existing building. Of course, the slab has a stiffening effect on the main beams.

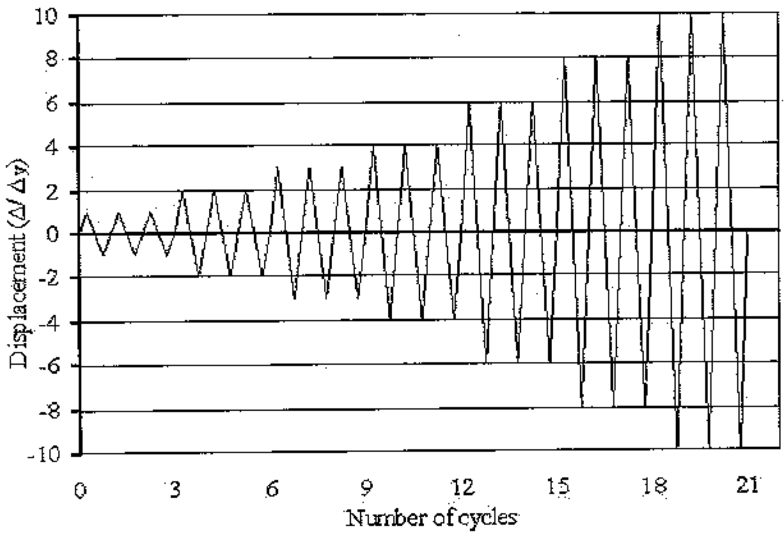


Fig. 16 Cyclic loading history

2.1 Reference Specimens

The details of the specimens are shown in Figure 17. Table 4 provides the material properties for the specimens. A concrete mix of 1:3.4:3.8 (cement: fine aggregate: coarse aggregate) with water-to-cement ratio of 0.75 was used. To avoid failure of the beams prior to that of the columns, both positive and negative yield moments of the beams were kept at least 10 percent higher than the target flexural capacity of the retrofitted columns. The members had adequate shear reinforcement to avoid any shear failure. Further, ties were provided in the joints to avoid any failure of the joints.

2.2 Retrofitted Specimens

The details of the specimens are shown in Figure 18. The fabrication of the reinforcement for the jacket is illustrated in Figure 19. The additional longitudinal bars were placed at the corners of the existing column. To continue the added bars, holes were drilled in the slab. Additional ties were not placed at the joint to avoid drilling of the beams. To stiffen the bars against buckling and to confine the joint region, angles were welded to the additional longitudinal bars at the joint. To minimize any reduction of ductility due to welding, bars from a reputed producer were used. To check the integrity of the welds, coupon specimens of bars with welded angles were tested under tension, and the necking and subsequent failure of the bars were found to be away from the welded segments. To enhance the integrity of the angles, those were tied together above the floor level and beneath the soffits of the beams with 10-mm diameter threaded bolts (see Figure 19(b)).

Each transverse bar for the jacket was made of two lapped U-bars. One of the U-bars was provided with 90° bends for additional lapping. The specimens were cast vertically to simulate the actual method of construction. The jackets of the retrofitted specimens were made of self-compacting concrete, as was done for the retrofitted column specimens.

3. Test Results

3.1 Reference Specimens

For the specimen tested under monotonic lateral loading, a plastic hinge was formed at the bottom column near the joint, followed by the buckling of longitudinal bars. For the specimen tested under cyclic lateral loading, plastic hinges were formed at both top and bottom columns near the joint, followed by the buckling of the longitudinal bars of the bottom column. The close-up views of the plastic hinges are shown in Figure 20. There was no distress in the beams or in the joints. The specimen showed stable hysteresis loops with the ultimate strength being reached at the 21st cycle.

3.2 Retrofitted Specimens

The columns in the retrofitted specimens were found to behave satisfactorily with regard to strength and ductility, without any premature failure, such as buckling of the additional longitudinal bars at the joint. There was no distress in the beams or in the joints. There was no shear crack in the columns. From this it can be inferred that the added ties made of U-bars were adequate in the present study. The close-up views of the plastic hinges in the columns for the specimen tested under cyclic lateral loading are shown in Figure 21.

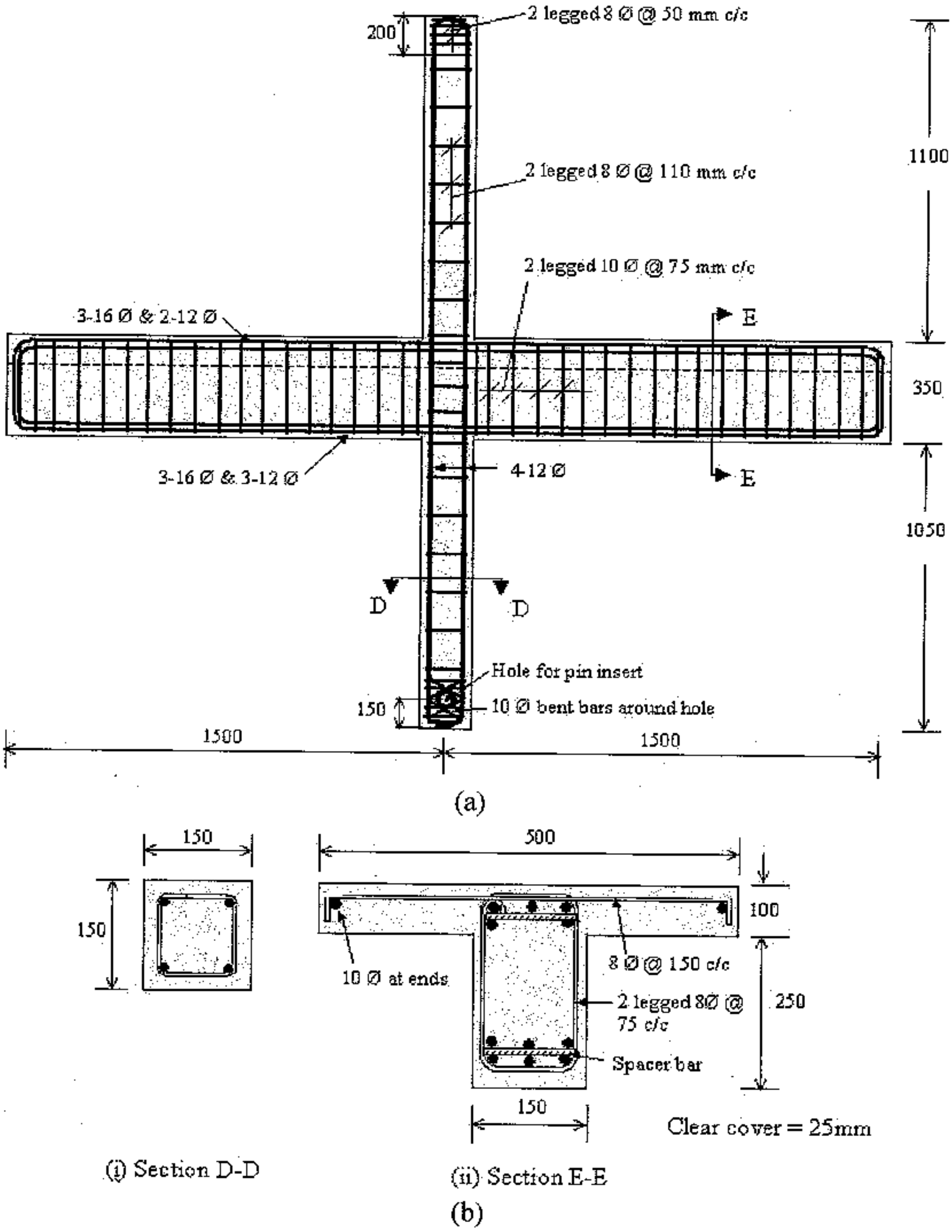


Fig. 17 Details of the reference specimen for beam-column-joint sub-assemblage tests (all dimensions are in mm): (a) reinforcement details for reference members, (b) cross-sections of members (not to scale)

Table 4: Material Properties for Sub-assemblage Specimens

Type of Lateral Loading	Reference			Retrofitted			
	f_{cm}	f_{yl}	f_{yt}	f_{cmE}	f_{cmJ}	f_{yl}	f_{yt}
Monotonic	24	435	468	22	31	483	504
Cyclic				24	32		

f_{cm} = mean cube strength of concrete (in MPa), f_{cmE} = mean cube strength of existing concrete (in MPa), f_{cmJ} = mean cube strength of jacket concrete (in MPa), f_{yl} = yield strength of longitudinal bars in the columns (in MPa), f_{yt} = yield strength of transverse bars in the columns (in MPa), modulus of elasticity for steel = 2.02×10^5 N/mm^2

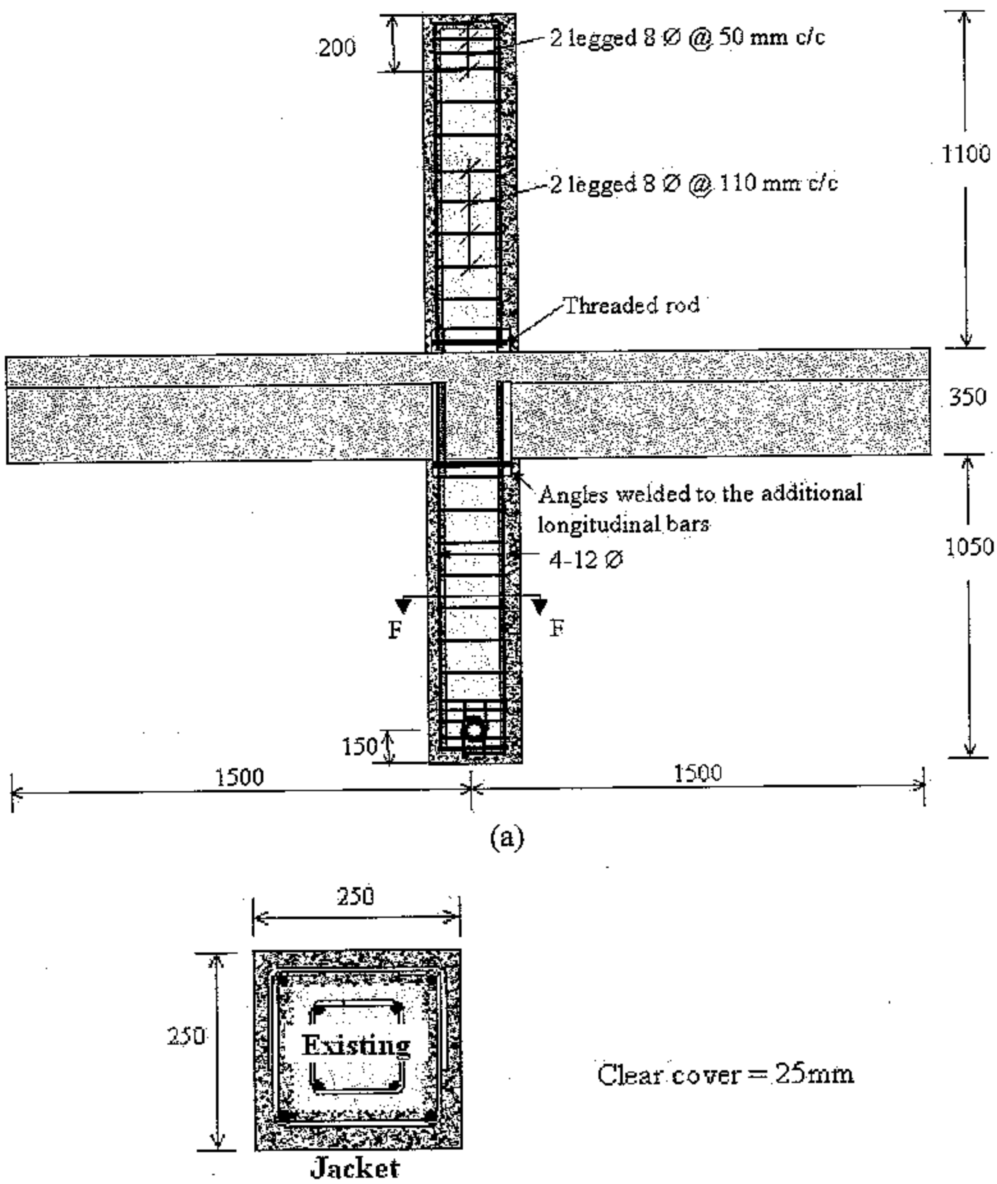
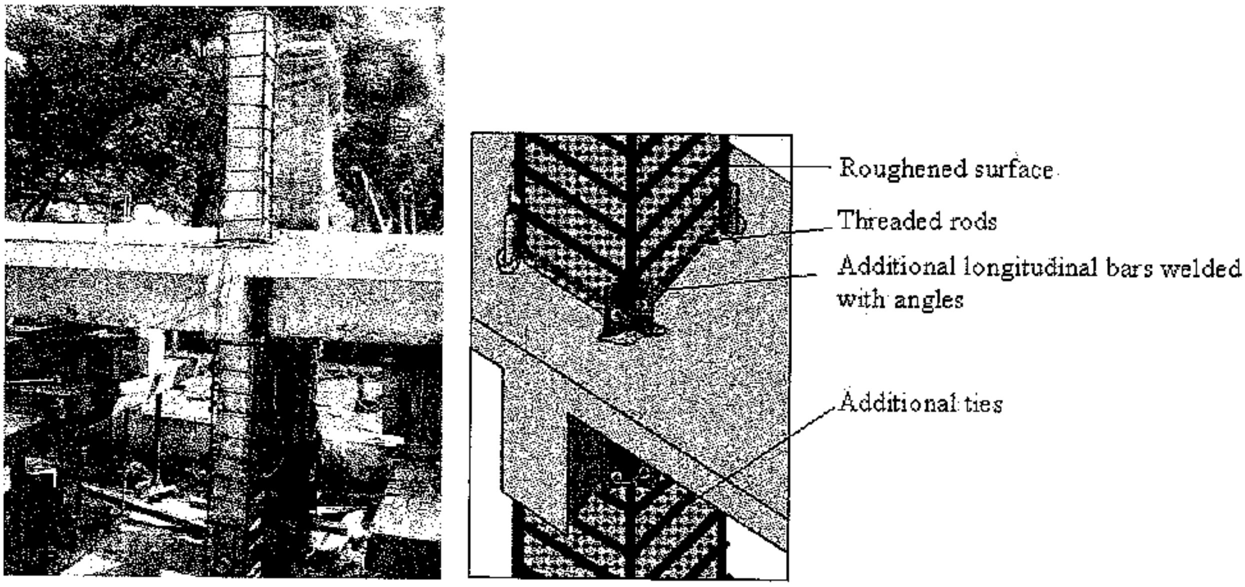
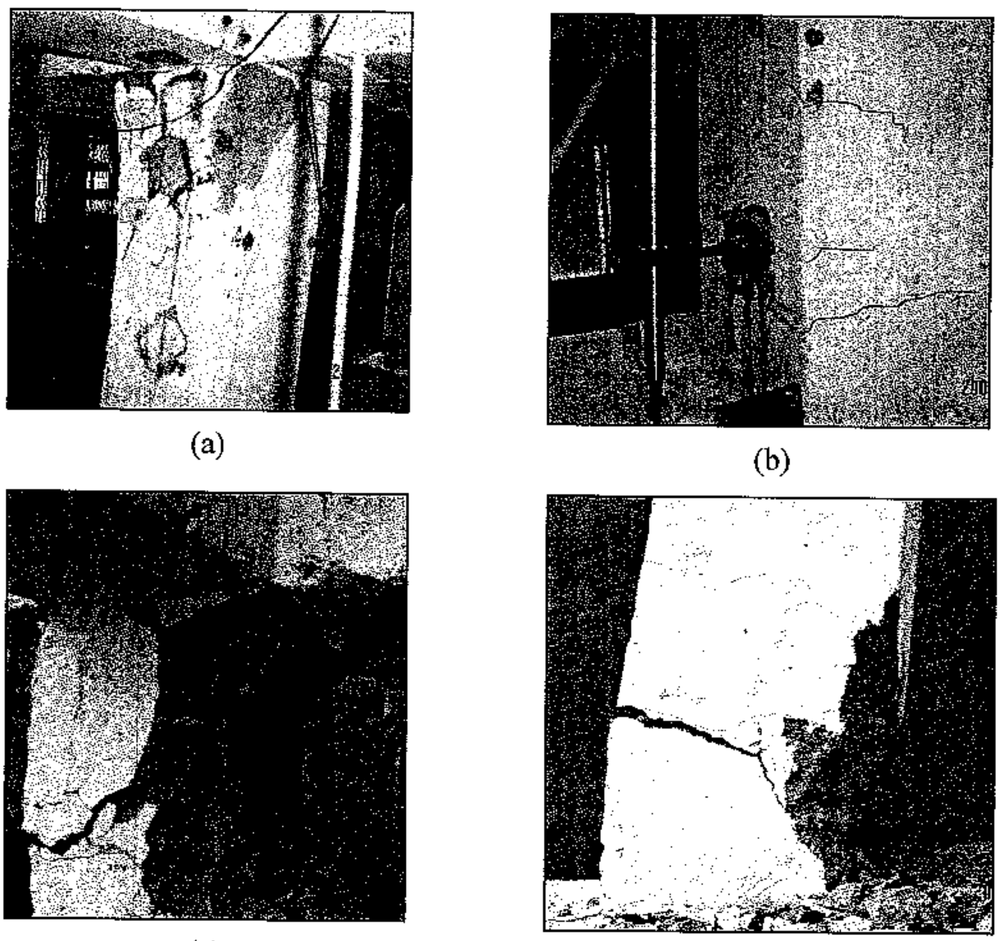


Fig. 18 Details of the retrofitted specimen for beam-column-joint sub-assemblage tests (all dimensions are in mm): (a) reinforcement details for retrofitted column, (b) Section F-F (not to scale) (these bars are in addition to those in the original cross-sections, which are same as in the reference specimen; see Figure 17)



(a) (b)

Fig. 19 Fabrication of reinforcement for retrofitted specimens: (a) photograph of fabricated reinforcement, (b) details at the beam-to-column joint



(a) (b) (c) (d)

Fig. 20 Reference specimens after testing: (a) bottom column under monotonic loading, (b) top column under monotonic loading, (c) bottom column under cyclic loading, (d) top column under cyclic loading

The measured lateral load versus displacement curves for the reference and retrofitted specimens tested under the monotonic lateral loading are compared in Figure 22. It may be observed that the lateral strength (equal to the maximum lateral load sustained) and the displacement ductility of the retrofitted specimen have improved substantially as compared to the reference specimen. The corresponding measured moment versus curvature curves for the hinges are shown in Figure 23.

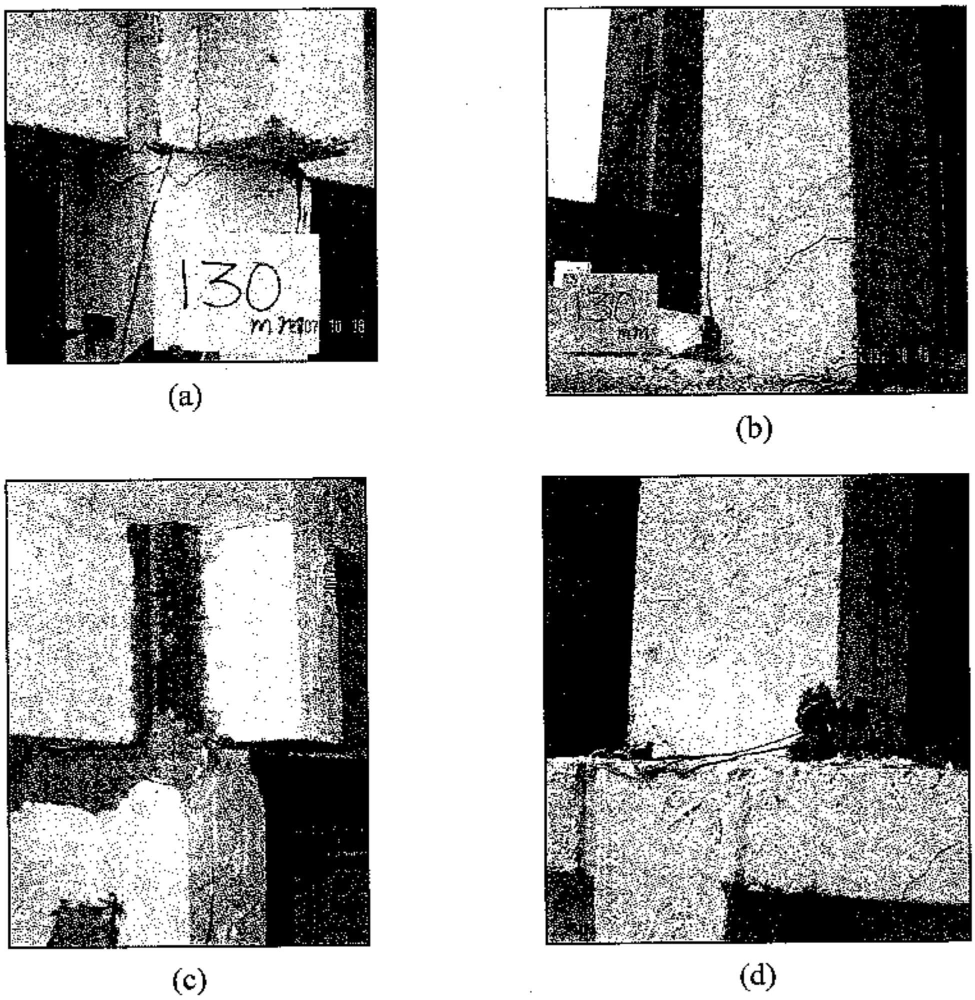


Fig. 21 Retrofitted specimens after testing: (a) bottom column under monotonic loading, (b) top column under monotonic loading, (c) bottom column under cyclic loading, (d) top column under cyclic loading

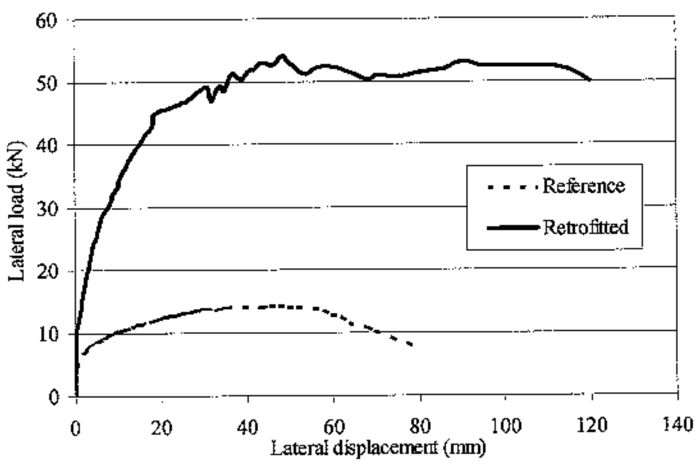


Fig. 22 Comparison of the lateral load versus displacement curves under monotonic loading

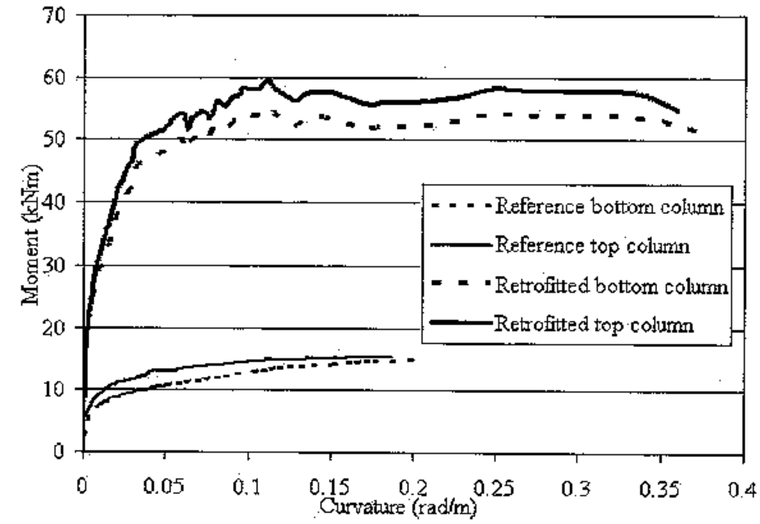


Fig. 23 Comparison of the moment versus curvature curves under monotonic loading

The lateral load versus displacement curves for the reference and retrofitted specimens tested under cyclic lateral loading are compared in Figure 24. The cumulative energy dissipated (based on the areas within the hysteresis loops) in the loading cycles by each of the reference and retrofitted specimens is shown in Figure 25. It may be observed that the energy dissipation has improved substantially due to jacketing.

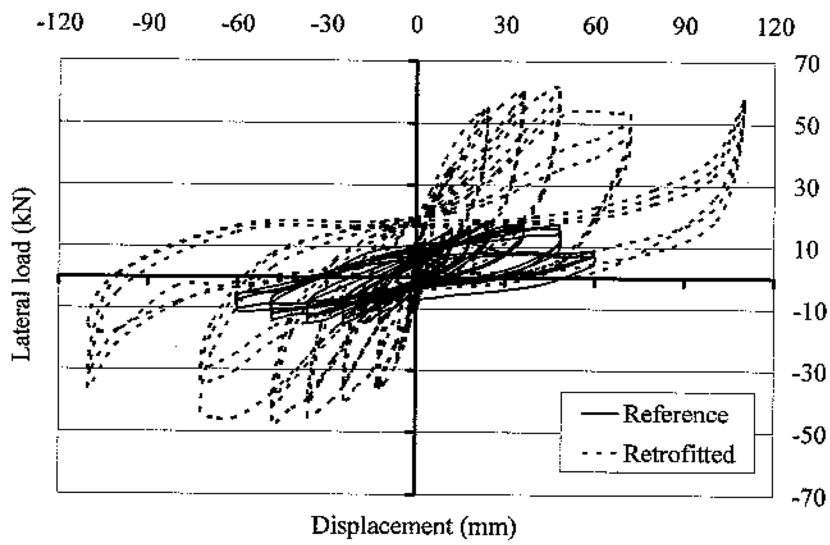


Fig. 24 Comparison of the lateral load versus displacement curves under cyclic loading

3.3 Comparison of Monotonic and Cyclic Tests

The lateral load versus displacement curves for the monotonic and cyclic loadings of the reference specimen are shown in Figure 26. The corresponding curves for the retrofitted specimen are shown in Figure 27. For both reference and retrofitted specimens, the monotonic tests provide sufficiently good estimates of the envelope of the lateral load versus displacement curves obtained under the cyclic loading. Hence, the values of ductility calculated from the lateral load versus displacement curves for monotonic loading are rational estimates.

3.4 Observations

The values of lateral strength and displacement ductility for the sub-assemblage specimens are given in Table 5. The values of lateral strength and lateral displacement for all specimens are compared in Figures 28 and 29, respectively.

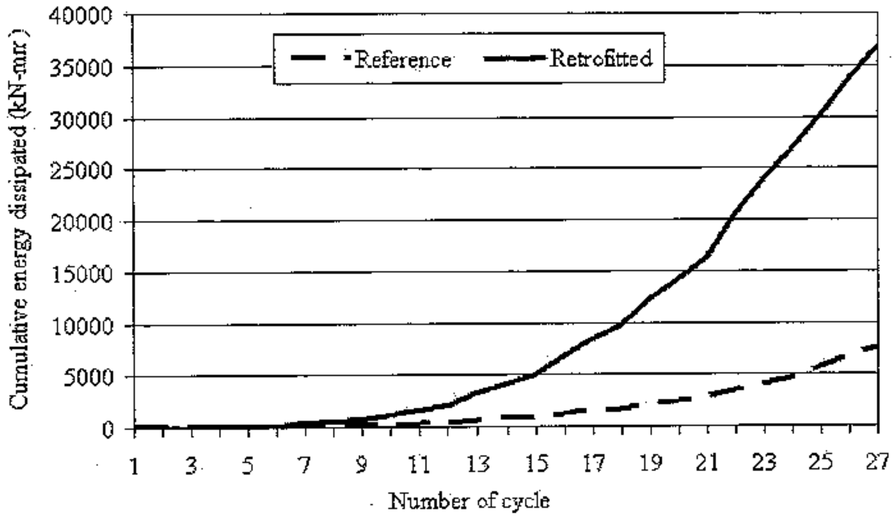


Fig. 25 Comparison of the curves for energy dissipated under cyclic loading

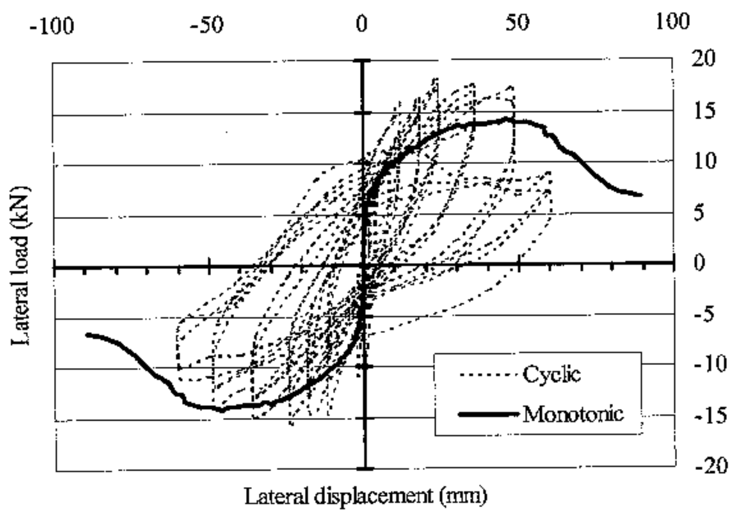


Fig. 26 Comparison of the lateral load versus displacement curves under monotonic and cyclic loadings for reference specimens

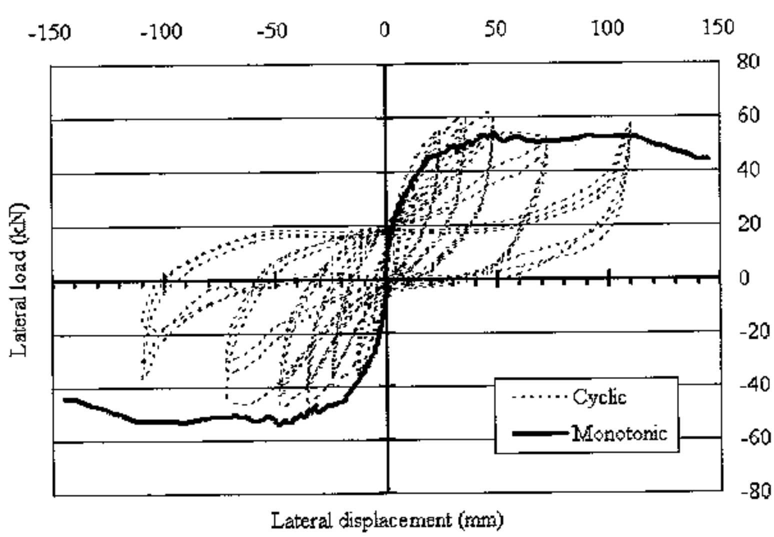


Fig. 27 Comparison of the lateral load versus displacement curves under monotonic and cyclic loadings for retrofitted specimens

Table 5: Values of Lateral Strength and Ductility for Sub-assemblage Specimens

Type of Lateral Loading		Type of Specimen	Lateral Strength (kN)	Lateral Displacement (mm)		Displacement Ductility Δ_u/Δ_y	Cumulative Energy Dissipated till 27th Cycle (kN-mm)
				Yield* Δ_y	Ultimate† Δ_u		
Monotonic		Reference	14	18	45	2.5	Not applicable
		Retrofitted	53	24	110	4.6	
Cyclic	–	Reference	14	30	48	1.6	7617
	+		17	24	48	2.0	
	–	Retrofitted	47	22	110	5.0	36944
	+		58	20	110	5.5	

*Displacement corresponding to a strain of $f_{yl}/E_{sl} + 0.002$ in the extreme longitudinal bars

†Displacement corresponding to the onset of drop in the applied lateral load

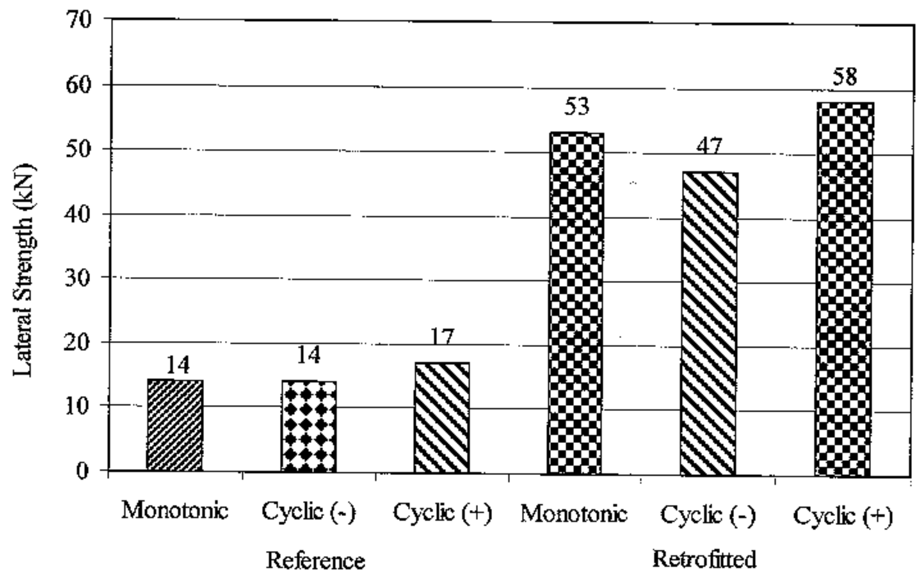


Fig. 28 Comparison of lateral strengths for reference and retrofitted specimens

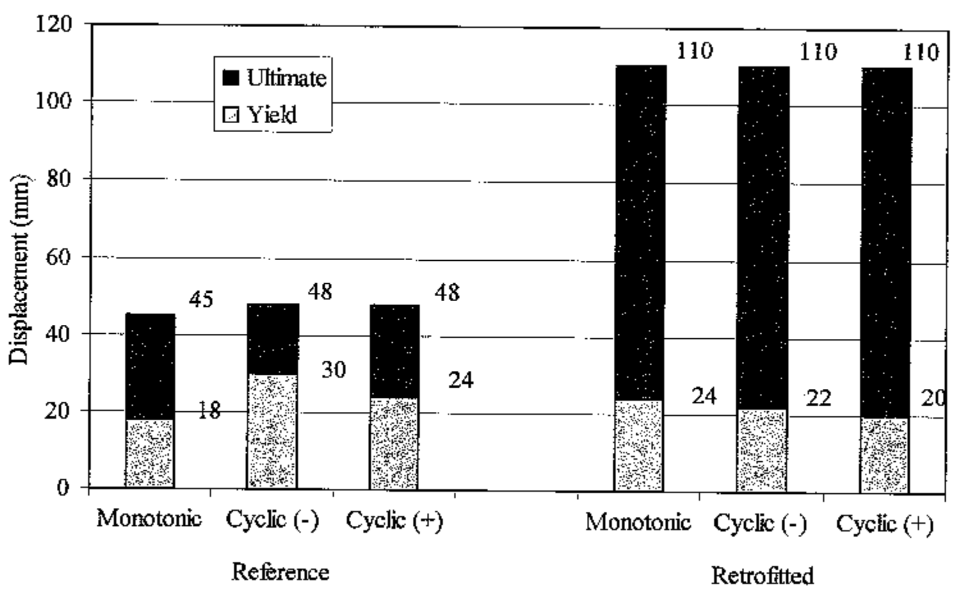


Fig. 29 Comparison of lateral displacements for reference and retrofitted specimens

The observations from the sub-assemblage tests are summarized below:

1. The retrofitted specimens did not show any visible delamination of the concrete in the jacket. The delamination was inspected by light tapping of a hammer. The sound due to any gap generated between the concrete layers and any chipping off the jacket concrete would have revealed delamination.
2. (a) The lateral strength of the retrofitted specimen tested under monotonic loading was 3.8 times higher than that of the reference specimen.
   (b) The lateral strengths (for both positive and negative cycles) of the retrofitted specimen tested under cyclic loading were about 3.3 times higher than those of the reference specimen.
3 (a) The displacement ductility of the retrofitted specimen tested under monotonic loading was 1.8 times higher than that of the reference specimen.
   (b) The values of displacement ductility of the retrofitted specimen tested under cyclic loading were 2.8 and 3.1 times higher than those of the reference specimen for the positive and negative cycles, respectively.
4. Under the cyclic loading, the cumulative energy dissipated till attaining the strength for the retrofitted specimen was 4.85 times higher than that for the reference specimen.

ANALYSIS OF RETROFITTED SECTIONS

The axial load versus moment interaction curves and moment versus curvature curves for the retrofitted sections are predicted by a lamellar (or layered) analysis and a simplified analysis.

1. Lamellar 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 lamellar method of analysis was used, wherein the section was divided into layers through the depth. A perfect bond was assumed for strain compatibility between the existing concrete and the jacket. The existing concrete was considered to be confined by the closely spaced added ties in the jacket. Among the available stress versus strain models for confined concrete, the model proposed by Mander et al. (1988) for confined concrete was selected. A parabolic and plastic stress versus strain model was used for the unconfined concrete of the jacket. The possible tension stiffening by the concrete in between the cracks prior to the yielding of the longitudinal bars was not considered in the sectional analysis.

2. Simplified Analysis

This analysis is preferred in the professional practice. This was conducted based on a uniform compressive strength (equal to that of the existing concrete) throughout the section. The stress versus strain curve as given in IS 456 (BIS, 2000) was used for the homogeneous concrete.

3. Results for Axial Load versus Flexural Strength Curves

The axial load versus moment interaction curves for the column specimens were developed by both of the above methods of analysis while satisfying the equilibrium conditions of axial force and moment at a section. The depth of neutral axis was varied to get a set of axial load and moment values. The material safety factors for steel and concrete were not considered.

Figure 30 shows comparison of the axial load versus moment interaction plots of the retrofitted columns as obtained from the tests and the predictions. The lamellar analysis shows good predictions of the strengths, except for Specimens PCR 1 and ECR 2. As mentioned earlier, these specimens had premature failures outside the test regions.

The simplified analysis provides lower strengths as compared to the tests, except for Specimens ECR 1, ECR 2, and ECR 3. Since a lower grade of concrete was used in determining the strength of a section, the predicted strength is expected to be lower than the experimental value. For Specimens ECR 1, ECR 2 and ECR 3, the compressive strength f_{cmE} of the existing concrete turned out to be more than the compressive strength f_{cmJ} of the concrete in the jackets because of the different periods of curing and trial mixes of the self-compacting concrete. The use of the compressive strength f_{cmE} of the existing concrete throughout the sections led to an overestimation of the strengths.

4. Results for Moment versus Curvature Behaviour

The moment versus curvature curves for the columns in the sub-assemblage specimens were developed by both lamellar and simplified analyses. The compressive strain in the extreme fibre of a section was varied to get a set of moment and curvature values. For the lamellar analysis, the failure strain for the concrete in the jacket was taken equal to the value measured from the test. For the simplified analysis, the corresponding failure strain was taken as 0.0035, as per IS 456 (BIS, 2000). The curves for the column section of the retrofitted sub-assemblage tested under the monotonic loading are shown in Figure 31. It may be observed that the lamellar analysis provides good prediction of the behaviour. The simplified analysis underestimates the strength, as the grade of the inner concrete was considered to be applicable throughout the section and the effect of confinement was neglected. Also, the analysis cannot predict the deformation beyond a certain value due to the limiting strain of 0.0035.

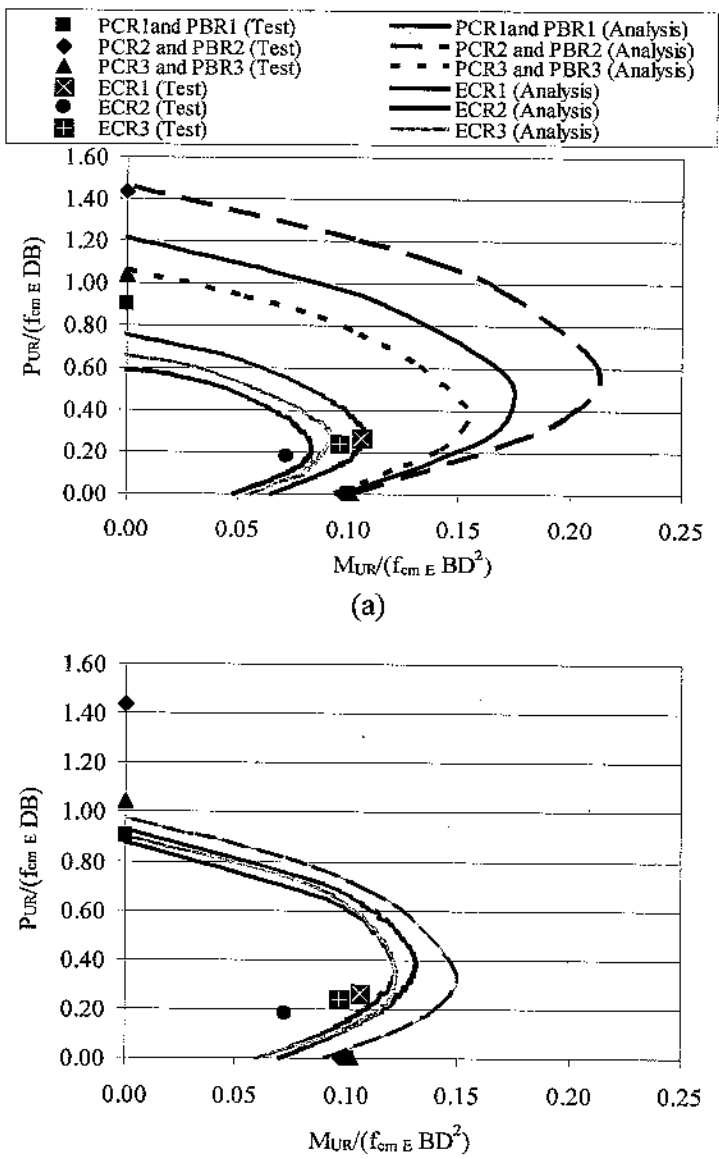


Fig. 30 Axial load versus moment interaction plots for retrofitted columns based on (a) lamellar analysis and (b) simplified analysis

5. Results for Lateral Load versus Displacement Behaviour

The lateral load versus displacement curve for the retrofitted specimen tested under monotonic lateral loading is predicted by a computational model of the sub-assemblage. Initially, a pushover analysis as per the software SAP 2000 NL was conducted. However, it was observed that the nonlinearity in the behaviour could not be predicted with sufficient accuracy by assigning the limited bilinear form (up to the peak) to the moment versus rotation hinge property. Next, an incremental nonlinear analysis was conducted with varying rotational stiffness for the plastic hinge regions of the columns. Only the latter analysis is presented here for brevity.

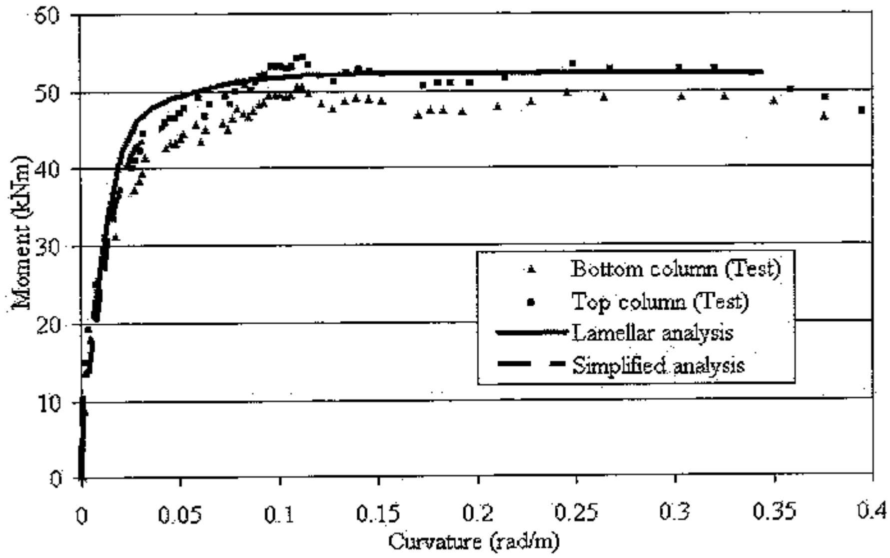


Fig. 31 Moment versus curvature curves for the retrofitted column section tested under monotonic loading

The computational model for the sub-assemblage developed using frame elements and appropriate boundary conditions is shown in Figure 32. To consider the spread plasticity, the top and bottom column members were sub-divided for isolating the plastic hinge regions of height $0.5D$ (where D is the overall depth of the column) from the faces of the joint. At each step P_{Hi} of the lateral load, the flexural stiffness, based on the predicted moment versus curvature values from a lamellar analysis of the column section, was assigned to each plastic hinge member. From the bending moment M_i (corresponding to P_{Hi}) in the column at the face of the joint, the moment of inertia I_i for a plastic hinge member was calculated based on the secant flexural stiffness E_cI_i of the predicted moment versus curvature curve (see Figure 33), where E_c is the modulus of concrete. Thus, $I_i = M_i/\phi_i E_c$, where ϕ_i is the curvature corresponding to M_i. Next, a linear analysis was performed to get the lateral displacement Δ_i at the top of the upper column due to P_{Hi}. The P-Δ effect due to the displacement of the vertical load P_V at the top of the upper column was included. The analysis was conducted for incremental values of P_{Hi}.

In the experiments, horizontal frictional forces were induced at the bearings at the ends of the beams and at the top of the upper column due to reactions from the axial load. These frictional forces increased the stiffness of the experimental lateral load versus displacement curve. For a precise prediction of the behaviour, it was decided to include the frictional forces in the computational model. First, the bearings were tested to determine their coefficients of friction. From the tests, it was found that the roller-cum-rocker bearings at the beam ends had a coefficient of friction, μ_r, equal to 0.11. The coefficient of friction, μ_s, for the sliding-cum-rocker bearing at the top of the column was 0.035. Next, frictional force at a bearing was 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 was calculated from statics due to P_V or P_H or both.

In Figure 32, the horizontal frictional forces at the beam ends and at the top of the column are denoted as $R_{F,A}$, $R_{F,B}$, and $R_{F,C}$, respectively. The corresponding vertical reactions are denoted as R_A, R_B, and

R_C, respectively. Additional forces corresponding to $R_{F,A}$ and $R_{F,B}$ were applied in the computational model to incorporate the effect of friction in the bearings at the beam ends. The value of $R_{F,C}$ was deducted from the applied P_H to incorporate the effect of friction in the bearing at the top of the upper column.

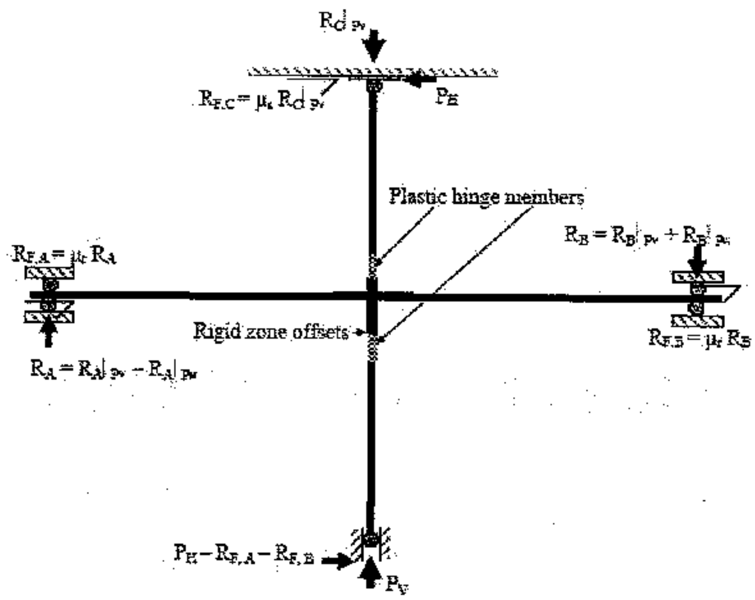


Fig. 32 Computational model for beam-column-joint sub-assemblage

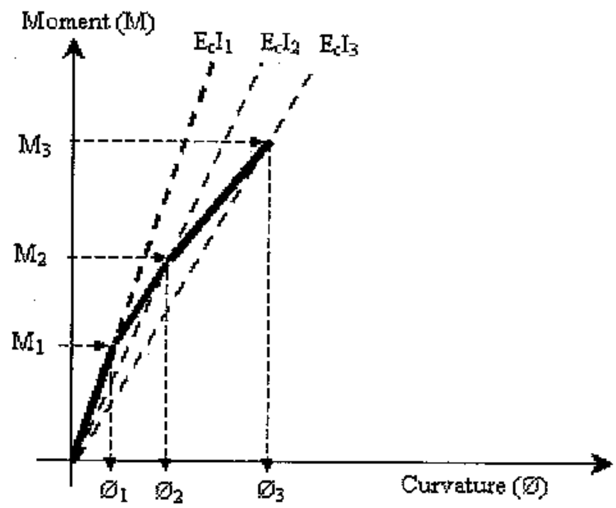


Fig. 33 Secant flexural stiffness from moment versus curvature curve

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 34. It may be observed that the predicted curve is close to the experimental results.

GUIDELINES FOR PROFESSIONAL PRACTICE

1. Analysis of Retrofitted Sections

A lamellar 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. On neglecting slippage, the strain difference at the interface of the two concretes 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.

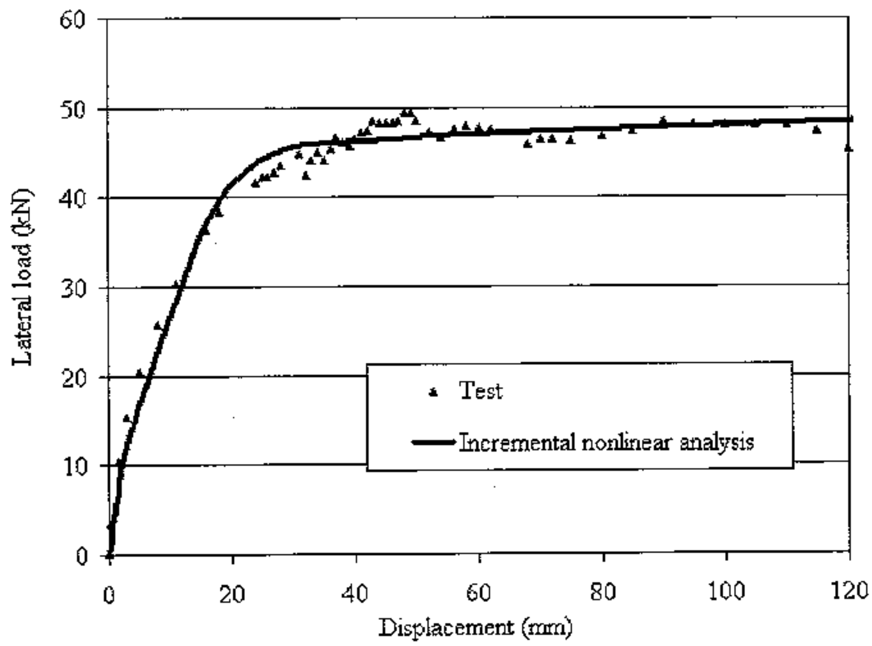


Fig. 34 Lateral load versus displacement curves for the retrofitted sub-assemblage tested under monotonic loading

2. Design of Additional Bars

The required additional longitudinal bars for a column are to be calculated based on the moment demand from the worst combination of gravity and seismic loads.

The additional ties are to be calculated based on the shear demand from the capacity-based design and the requirement of confinement (BIS, 1993).

3. Detailing

To avoid any damage to the primary frame members of poor concrete, drilling of holes should be minimized in selecting a scheme of reinforcement for the concrete jacket. The additional longitudinal bars can be placed near the corners of a joint. This will avoid drilling holes through the beams. The bars should continue through the holes drilled in the intermediate floor slabs and be anchored into the footing. A minimum cover should be provided as per IS 456 (BIS, 2000) to satisfy the fire safety and durability requirements.

Additional ties for the jacket should be closely spaced in the plastic hinge regions, as per the ductile detailing requirements of IS 13920 (BIS, 1993). The ties should be provided preferably with 135° end-hooks, if the thickness of the jacket is large. Else, the ties can be made of U-bars with adequate lap length or welded lap. Of course, additional tests are required with the shear-dominated behaviour of the columns in order to study the performance of the U-bars.

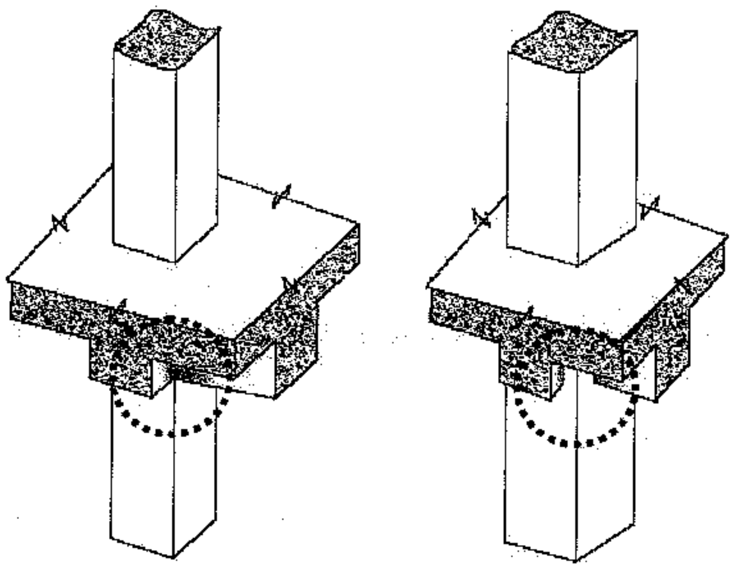
When a joint in the existing building does not have any ties, additional ties may be provided to increase the shear capacity of the joint. However, this involves drilling of the supported beams. Alternatively, to enhance the confinement of concrete at a joint, angles or cruciform sections can be provided at the corners of the joint and properly tied with threaded rods at the top and bottom of the joint. Following are some recommendations for this scheme:

- The surfaces of the angles or the cruciform sections should be roughened by using scribers, for better bond with the concrete surface.
- The additional longitudinal bars should be welded or clamped to the angles/cruciform sections by using steel loops at frequent intervals. For welding, weldable bars with adequate ductility should be

used. Also, the quality control specifications regarding selection of electrodes and workmanship should be adhered to.

Two typical interior joints are shown in Figure 35. The cross-sectional details near the joints are illustrated in Figure 36.

For a column wider than 300 mm, distributed longitudinal bars along the surfaces should be provided. These bars are not continuous at the joint. However, they should be tied to the added ties and provided with end hooks. This tying of bars will also reduce the unsupported lengths of the added ties.



(a) (b)

Fig. 35 Typical interior beam-to-column joints (the reinforcement bars are not shown): (a) width of column same as that of beams, (b) width of column larger than that of beams

4. Construction

If possible, the existing load in a column can be reduced by providing adjacent temporary supports and adequate shoring before constructing the jacket.

After chipping any plaster, the surface of the existing concrete should be roughened without damaging the concrete, especially due to poor quality of the existing concrete. Hence, roughening by a motorized wire brush is preferred to hacking. Sand blasting the surface can provide more roughness by exposing the aggregates.

Self-compacting concrete can be used for the jacket. The flowability of concrete is important with regard to the filling up of the annular space between the existing column and the formwork. A shrinkage-compensating admixture needs to be added to the new concrete.

The concrete jacket should be adequately cured before the column is reloaded.

SUMMARY AND CONCLUSIONS

The present study has investigated the effect of jacketing on the flexural strength and performance of columns. First, the specimen details and results of the slant shear tests for studying the interface between old and new concrete have been presented. Second, the testing of reference and retrofitted columns under pure compression, eccentric compression and pure bending has been documented. Third, the setup and testing of beam-column-joint sub-assemblage specimens have been explained. Next, the prediction of the strength and behaviour of retrofitted column specimens has been illustrated. A lamellar analysis and a simplified analysis were used for the prediction. The incremental nonlinear analysis for predicting the lateral load versus displacement behaviour of a sub-assemblage has been highlighted. Finally, guidelines for the retrofitting of columns by concrete jacketing have been provided.

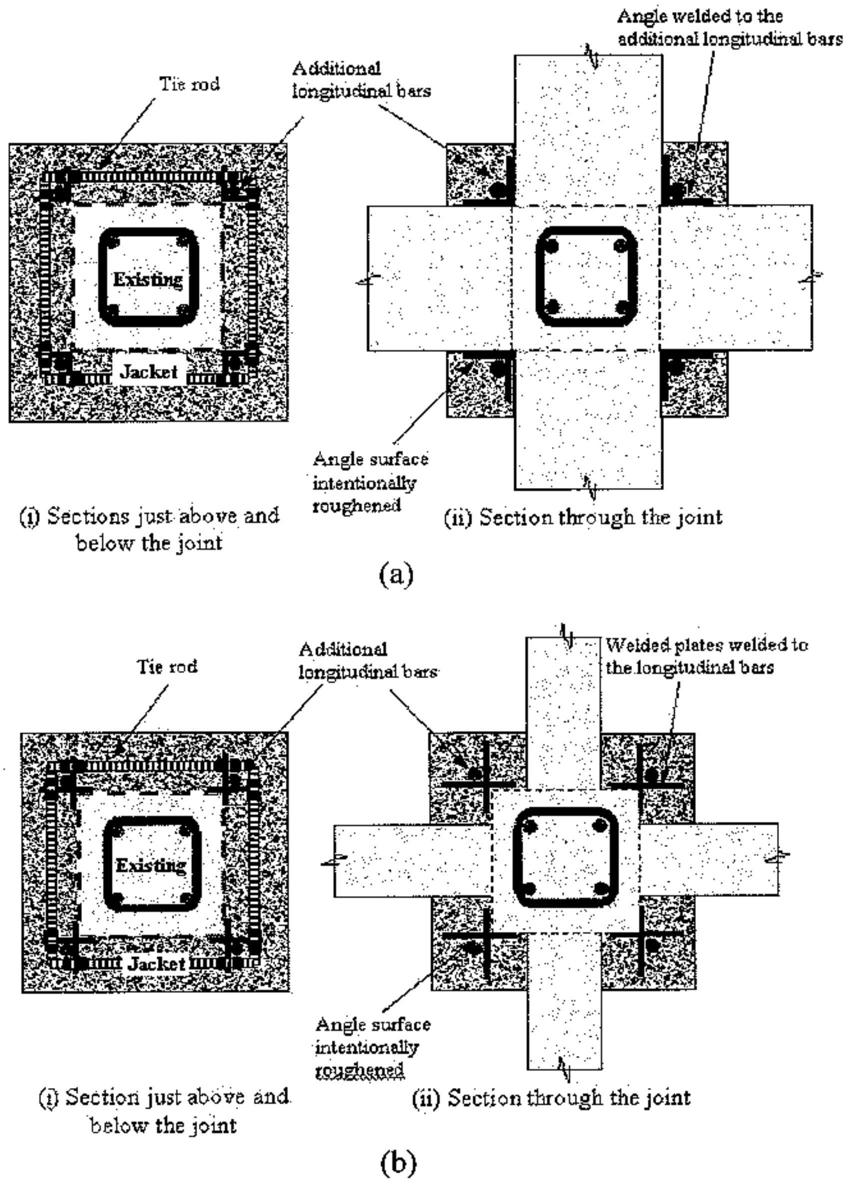


Fig. 36 Details of enhancing confinement of beam-to-column joints: (a) width of column same as that of beams, (b) width of column larger than that of beams

Following are the conclusions from the present study:

- The self-compacting concrete was found to be suitable for use in the concrete jacket.
- The retrofitted specimens did not show any visible delamination between the existing concrete and the concrete in the jacket. The roughening of the surface of the existing concrete by motorized wire brush was found to be satisfactory for the type of tests conducted.
- The moment capacities of the retrofitted column specimens were substantially more than those of the existing columns. This increase in capacities could be predicted by analysis.
- The retrofitted beam-column-joint sub-assemblage specimens showed substantial increase in lateral strength, ductility (i.e., energy absorption) and energy dissipation.
- The degradations in strength and stiffness of the retrofitted sub-assemblage specimen tested under cyclic loading were limited.
- A lamellar analysis considering the two grades of concrete in a retrofitted section, and the effect of confinement on the stress versus strain curve for concrete under compression, provides a good prediction of the strength and the moment versus curvature behaviour of the section. However, a simplified analysis considering the lower grade of concrete for the whole section and using the code-specified stress versus strain curve for the concrete under compression can give a conservative value

of the strength alone. It cannot correctly predict ductility in the moment versus curvature behaviour of the section.

- The prediction of the lateral load versus displacement behaviour of a sub-assemblage in a building by the pushover analysis using bilinear (up to the peak) moment versus rotation curve for a plastic hinge is approximate, especially in the pre-yield region.
- The incremental nonlinear analysis, with varying flexural stiffness for the hinge members (included to model the spread plasticity) and incorporating friction of the bearings, showed substantially better prediction of the lateral load versus displacement behaviour of the retrofitted sub-assemblage specimen as compared to the pushover analysis. The tension stiffening effect of cracked concrete may be considered for improved predictions in the pre-yield region.

Regarding the retrofitting of columns for flexure, tests can be conducted on larger-scale specimens with reduced increase in area after jacketing to study the improvements in strength and performance. The scheme of concrete jacketing selected in the present study needs to be qualified under a fast cyclic loading. This study can be extended to the exterior or corner columns by testing the corresponding sub-assemblage specimens. Three-dimensional frames with jacketed columns can also be tested under the monotonic or cyclic lateral loads, or under a base excitation by using a shake table.

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