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REHABILITATION OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

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ABSTRACT

An innovative seismic rehabilitation technique for existing reinforced concrete connections using corrugated steel jacketing was proposed and evaluated. The seismic performance of four one third scale, reinforced concrete beam-column connections were studied. The connections include one existing, the second is designed according to current seismic codes and two rehabilitated connections. The corrugated steel jacket provides both lateral confinement and shear reinforcement. The experimental results indicate that existing reinforced concrete frame connections rehabilitated using corrugated steel jackets performed satisfactorily under high cyclic load levels.

KEYWORDS

Reinforced concrete; experimental; steel jacketing; joints; seismic; rehabilitation.

INTRODUCTION

Existing structures that were designed according to earlier codes may not meet current seismic design standards. Many are inadequate and pose a severe risk during seismic events. The rehabilitation of existing structures is gaining considerable interest. Vulnerable structures may be retroffited to assure compliance with current design provisions. In this study, a new innovative method of joint rehabilitation is proposed and evaluated. The method involves encasing deficient beam-to-column joints with a corrugated steel jacket and filling the gap with non-shrink grout as shown in fig. 1. This system is an efficient solution to joint upgrade as the corrugated steel provides both lateral confinement and shear reinforcement. In effect, the corrugated jacket adds shear strength and ductility to the connection. This approach can be applied to undamaged deficient structures and when appropriate, to frames that have been lightly damaged by an earthquake. Ideally, the beam-column joint, which is considered part of the column, should remain elastic, thus ensuring that plastic hinges are formed in adjacent beams according to the design philosophy of strong column/weak beam and strong shear/weak moment. Modern design codes address these concepts by specifying a ratio of strength between the beam and column of a joint.

A study of the behaviour and seismic rehabilitation of beam-column connections was conducted at McMaster University with the following objectives: (1) quantify the behaviour of existing beam-column connections under seismic loading, and estimate their performance during future earthquakes; (2) formulate rehabilitation design criteria and develop a rehabilitation technique that will improve seismic performance of the beam-column connections; (3) evaluate the performance of the rehabilitated connections experimentally and (4) develop rehabilitation design recommendations. The primary focus of this article is to present the behaviour of the existing and rehabilitated connections and compare the result with a connection detailed according to the current Canadian seismic code. An overview of the design of the jacket used for the connection rehabilitation is presented.

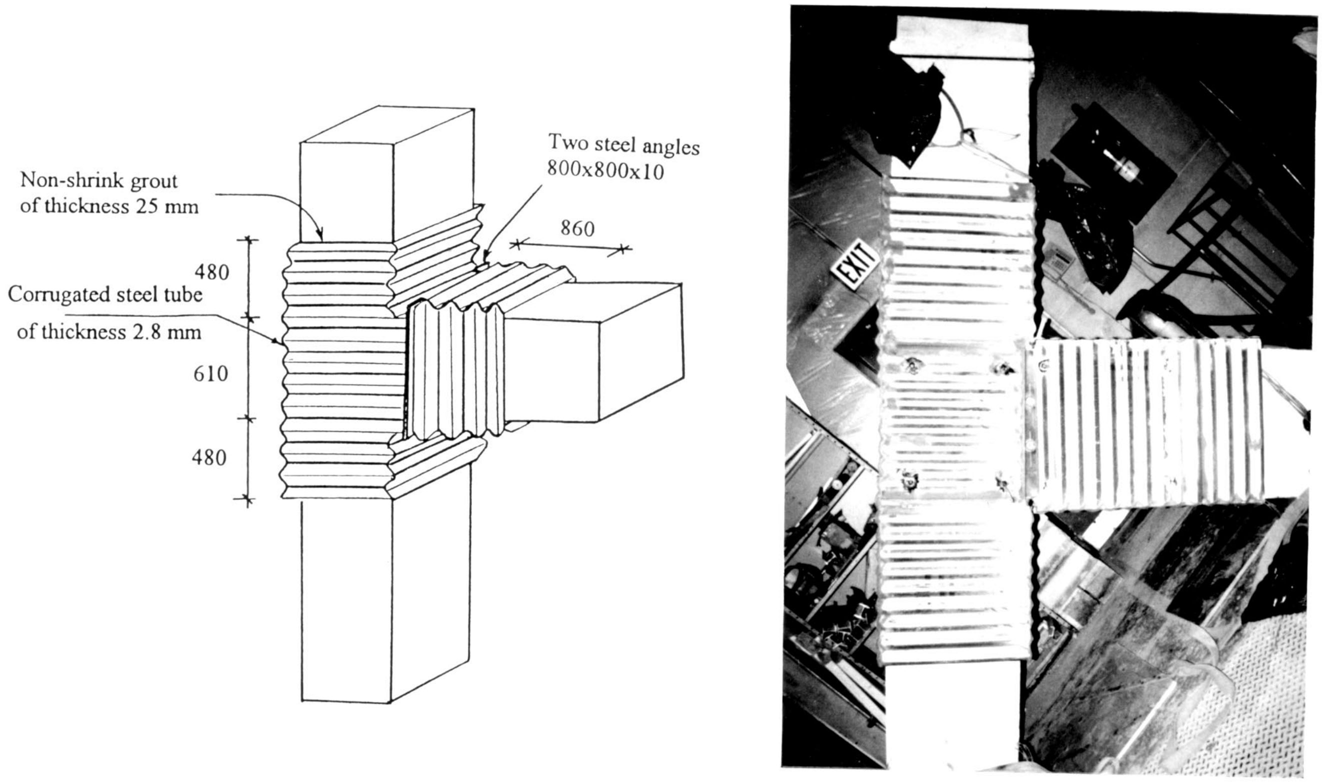


Fig. 1. The proposed rehabilitation technique.

ADVANTAGES OF CORRUGATED STEEL

When an ordinary steel jacket without corrugations is filled with grout and compressed, the apparent Poisson's ratio of the elastic steel jacket of 0.28 is larger than the Poisson's ratio of the infilled grout and concrete of 0.18. The infilled grout and the concrete do not bond to the steel jacket, thus lateral confinement will not be effective until the infilled grout and concrete become plastic and its Poisson's ratio increases beyond 0.28 and approaches 0.5, the value at the time of compression failure. However, the failure strain of concrete of 0.003 to 0.005 is larger than the yield strain of the steel jacket of 0.0018. Therefore, when Navier hypothesis can be assumed with regard to the infilled grout, concrete and steel jacket, the increment of compression load capacity of the infilled grout and concrete caused by the confining effect of the jacket is small enough to be neglected in structural design. The poor behaviour of columns confined by flat plates can be illustrated by the results of the experiments conducted by Priestly *et al.* (1994). In their investigation, a cyclic lateral load was applied on several columns with flat steel jackets and as a result outward bulging at midpoints of the flat plates was observed as shown in fig. 2. In the case of the corrugated steel jacket, however, the dimensions hardly change even when it is compressed in the axial direction and the apparent lateral expansion is almost nonexistent. Therefore, when the gap is filled with grout and compressed, lateral confinement effect is produced at the first stage of loading when the infilled grout and concrete are behaving elastically (Tomii, M., 1993). Since the axial rigidity of a corrugated steel jacket is small, the in-plan stress of the jacket wall is mostly the tensile stress in the ridge direction which contributes to lateral confinement. As a result, like high-strength hoops, a corrugated steel jacket is expected to produce a strong lateral confinement stress corresponding to the yield strength of the corrugated steel jacket. Therefore, compared with ties or steel jackets without undulation, a corrugated steel jacket has extremely large flexural rigidity with regard to the out of plane bulging.

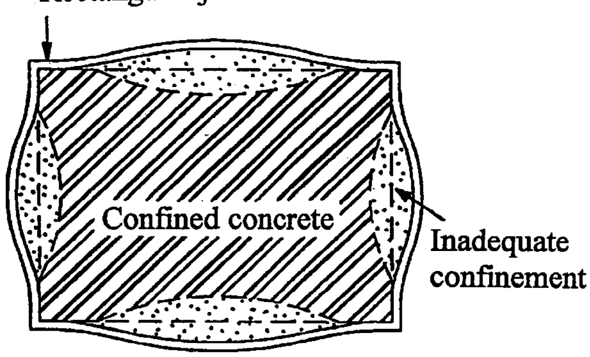


Fig. 2. Rectangular section confined by a flat rectangular tube (Priestly *et al.*, 1994)

EXPERIMENTAL PROGRAM

Test Description

The first floor exterior column connection of an existing frame structure, shown in fig. 3, was tested. The structure was designed and constructed in 1969 before current seismic codes were applicable. Because of laboratory space and equipment limitations, a one third scale test specimen was selected as shown in fig. 3. The criteria used in the modelling of specimens was discussed elsewhere (Biddah *et al.*, 1995). The test results described represent the first four specimens of an ongoing research program to determine the seismic capacity of existing frame structures and a suitable rehabilitation scheme for these structures. The four specimens have the same concrete dimensions. Each specimen consists of a beam 1.27 meters long, framing into a column 3.45 meters high at a distance of 1.27 meters from the top. The beam and column ends of the specimens are assumed to correspond to the approximate location of the inflection points of the prototype frame. The only difference between the specimens is the transverse reinforcement in the beam, column and joint. The first, third and fourth specimens, J1, J3 and J4, were detailed to represent the steel of the existing reinforced concrete frame. The second specimen, J2, was detailed according to the current Canadian seismic design code, (1994). The different reinforcement details of the specimens are shown in fig. 4. In the existing frame, the ties inside the joint and also in the column are about 16% of the steel recommended by the current Canadian seismic design code. Specimens J3 and J4 were encased by a corrugated steel jacket on the beam and column and on the column only, respectively, to enhance their seismic behaviour. The corrugated steel jacket was taken as 2.8 mm thick with a grouted gap of 25 mm thickness. Steel angles were attached to the beam at the column face to resist the outward confining pressure from the concrete in the joint part. Due to the insufficient shear reinforcement in the beam, a corrugated steel jacket was also used on the beam for specimen J3 as shown in fig. 1 with 20 mm gap from the critical section, (approximately representing the recommended 50 mm gap for full-scale connection). The purpose of the gap is to avoid unnecessary flexural strength enhancement which may upset the relative strength ratio of the connected beam and column.

Concrete in the existing structure was assumed to have a compressive strength of 21 MPa. The compressive strength of concrete cylinders at the day of testing was 23 MPa. Grade 400 reinforcing steel was used as the principal steel in all the specimens and as ties in specimen J2 with measured yield strength of 440 MPa. Smooth wire size rods were used for the ties in specimens J1, J3 and J4. The ratio of the factored resistance moment of the columns to the nominal resistance moment of the beam is 1.55 which is higher than the 1.1 ratio specified by the current Canadian code.

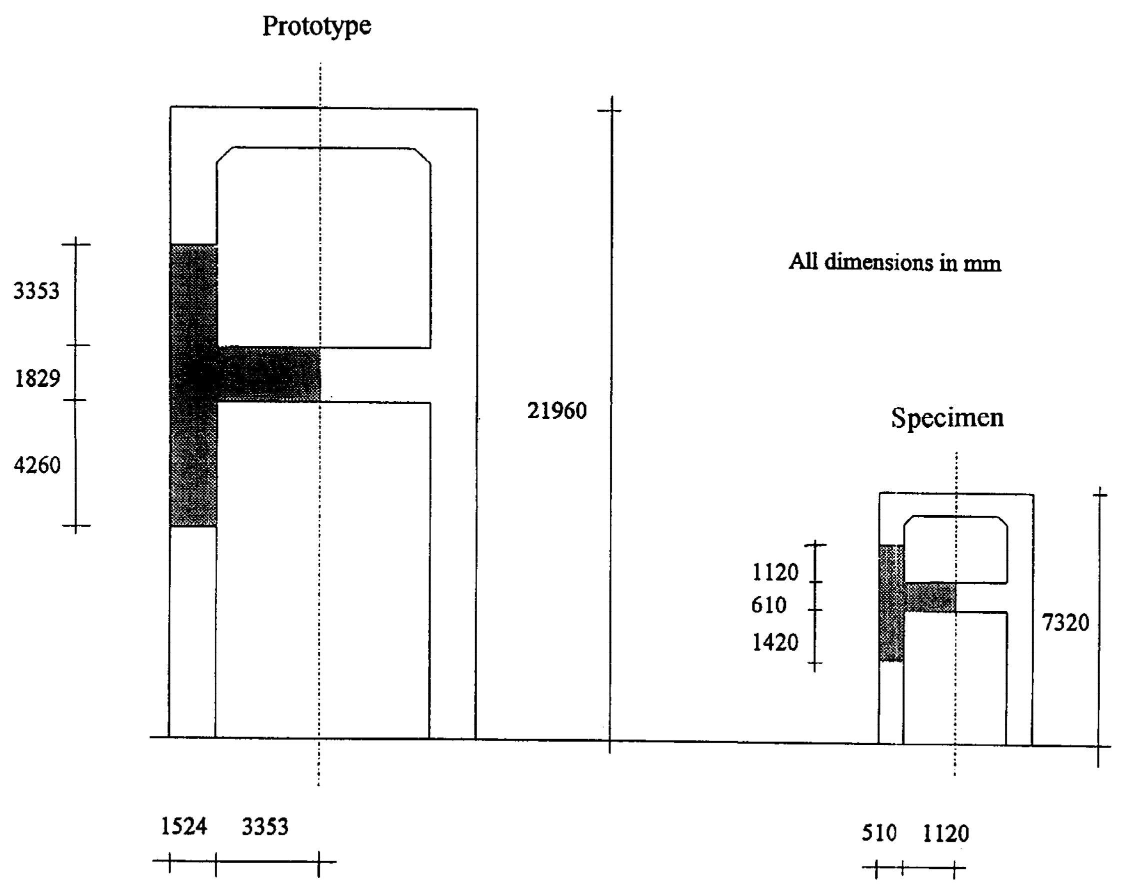


Fig. 3. Dimensions of the frame and one-third scale specimen.

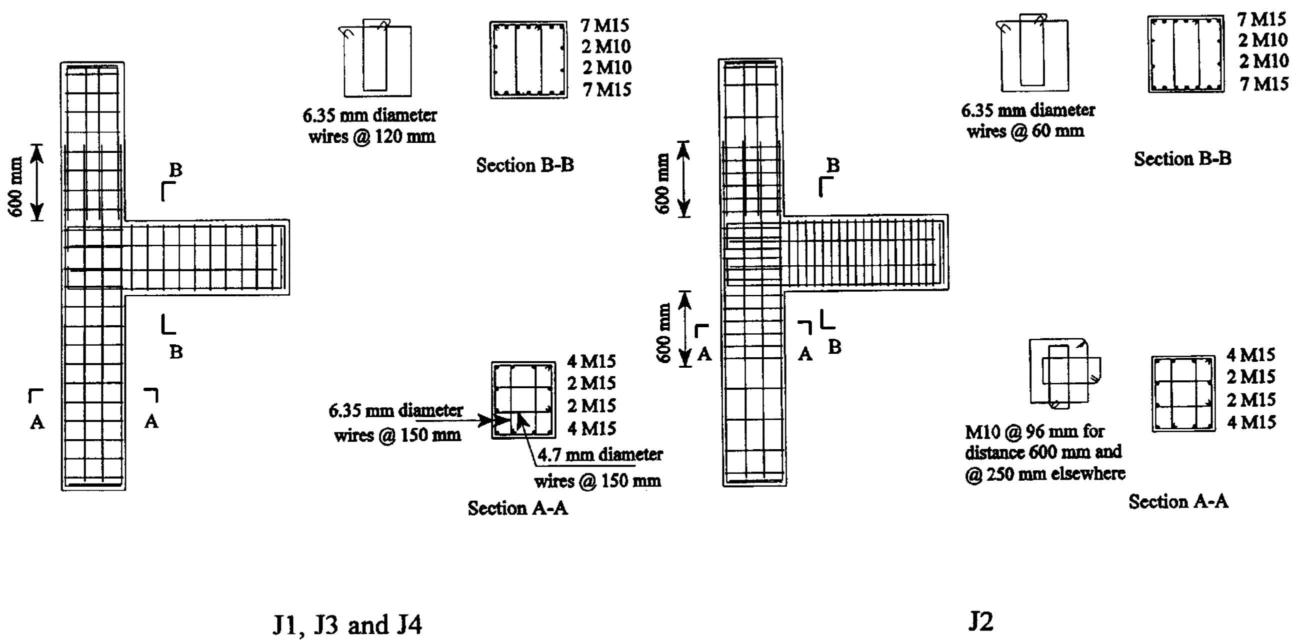


Fig. 4. Reinforcement details.

The Corrugated Steel Jacket

The corrugated steel section was selected to have the ratio of the pitch to maximum column width equals to 0.1 so that half the pitch would be almost equal to the thickness of the concrete cover of the main bars. The corrugated steel thickness was selected as 2.8 mm so that the area of the steel satisfies the transverse reinforcement requirement in the joint by the current Canadian seismic code and satisfies also the amount of confining steel required for a given curvature ductility factor depending on the level of axial load. The thickness of the non-shrink grout between the concrete and the steel jacket was selected as 25 mm so that the inertia of the composite section (corrugated steel and grout) should not be less than the stiffness of the minimum tie diameter at the maximum spacing.

It is proposed to build up the corrugated steel jackets for the column and beam from premanufactured "U" shape sections to be welded at the site as shown in fig. 5. However, for the joint jacket, two steel angles are proposed to resist the lateral pressure. these two angles are anchored to the concrete by expansion anchor bolts.

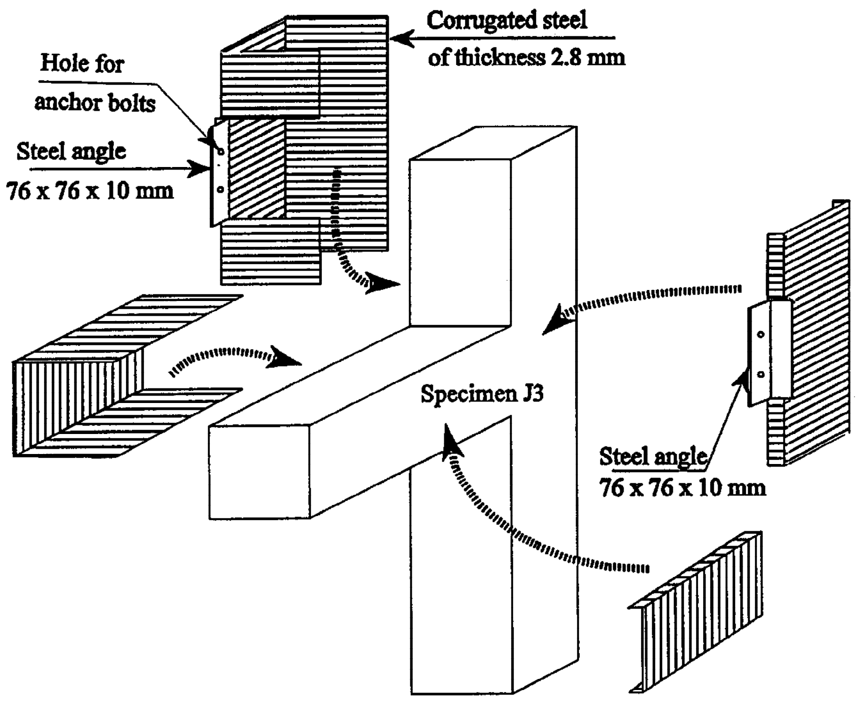


Fig. 5. Assembling of beam and column jackets.

Test Set-up and Instrumentation

The loading set-up for the test is shown in fig. 6. An axial load representing the gravity load, was applied to the column and kept constant throughout the test. Reversed cyclic displacements were applied to the free end of the beam as shown in fig. 6. Two cycles with same applied displacement were repeated before the displacement was further increased. The displacement schedule was intended to cause severe drifts to test the inelastic response of the specimens. Twenty four strain gauges were installed in the critical regions of the reinforcing steel of each specimen. Nineteen displacement transducers were used to separate and identify the various types of deformations that make up the total deformation of the beam-column subassemblage. All data from load cells, displacement transducers and strain gauges were recorded by a computer controlled data acquisition system.

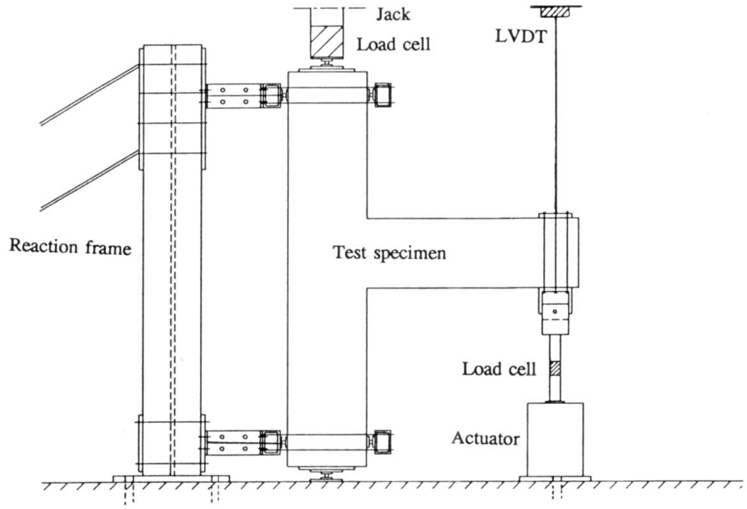
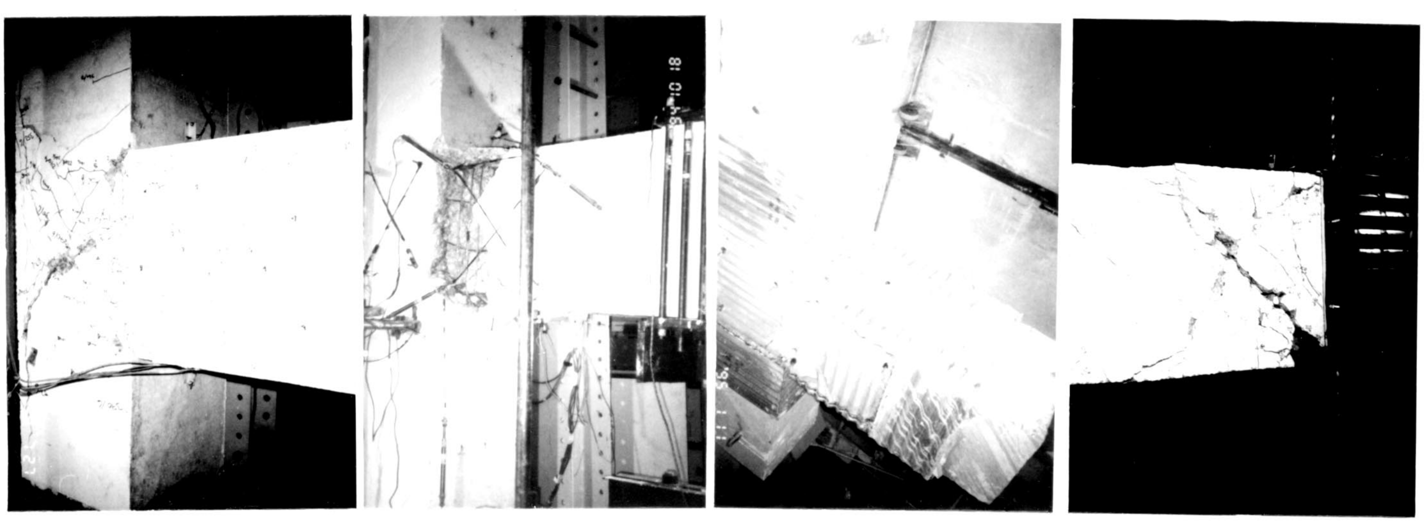


Fig. 6. Test set-up.

Test Results

Typical Sequence of Failure. During the tests, the cracking pattern was carefully marked and recorded. The first crack was in the beam at the face of the column for all the specimens. Specimen J1 showed beam flexural-shear cracks and then diagonal shear cracks appeared in the joint. By increasing the beam tip displacement, the joint shear capacity deteriorated. Although the anchorage length in specimen J1 satisfied the current Canadian code requirement, a bond failure of the beam reinforcement was reached at cycle 9. Specimen J2 showed a plastic hinge in the beam of length approximately equal to half the depth of the beam with almost no shear deformations occurring in the joint, also, no bond slip was observed. Specimen J1 was not capable of supplying the same anchorage capacity as specimen J2 because of the insufficient confinement of the joint of J1. Specimen J3 showed a plastic hinge in the beam between the beam jacket and the column face and almost no shear deformation in the joint. Specimen J4, with corrugated steel jacket on the column and joint only, protected the joint from large shear deformation. Shear failure in the beam plastic hinge occurred due to insufficient transverse reinforcement in the beam. Typical crack patterns of the specimens are shown in figure 7.



J1 J2 J3 J4

Fig. 7. Crack patterns in the specimens.

Hysteretic Behaviour. The overall behaviour of the specimens is shown using story shear-drift angle relationships. Drift angle, R, is defined as the beam tip displacement divided by the beam length. Story shear, V, is computed from the beam tip load, F, considering the P-Δ effect that may occur during an actual earthquake. The envelope of the story shear - drift angle for each specimen is shown in fig. 8. From the figure, jacketing of the frame elements was shown to improve the seismic behaviour of the existing structure as illustrated by the behaviour of specimens J3 and J4.

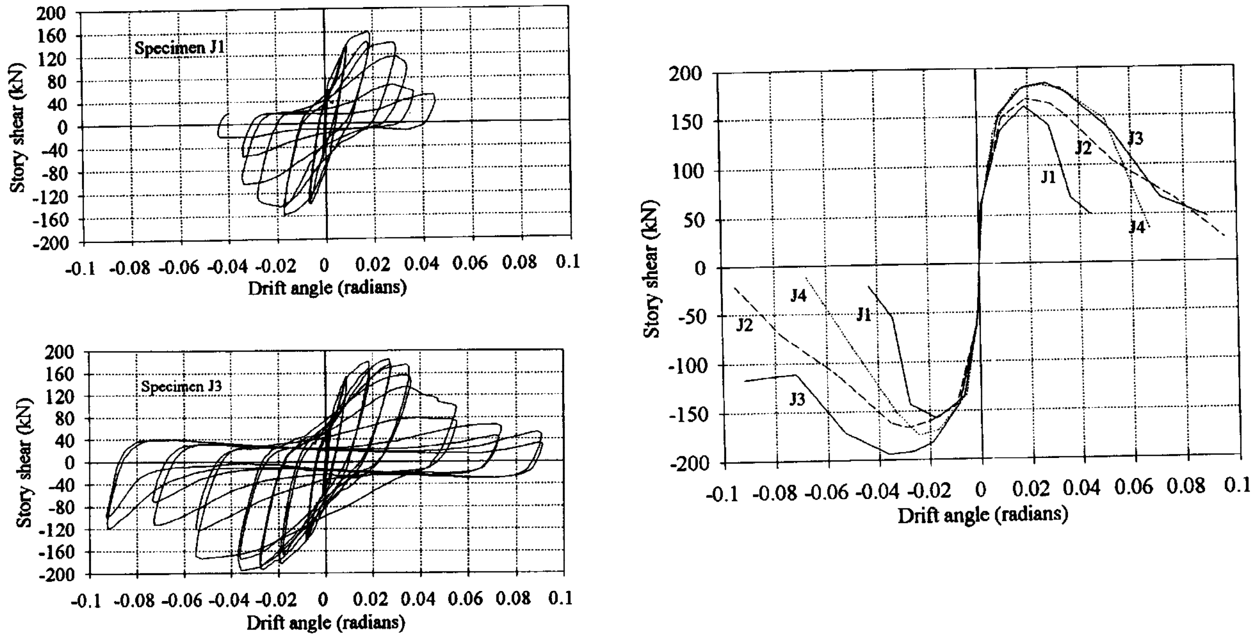


Fig. 8. Story shear-drift angle curves and envelope for the specimens.

Member Contribution to Drift Angle. Member contributions to drift angle for all specimens are outlined in fig. 9. Results are presented for the first negative cycle to given ductility level, i.e. ductility factors of 2 and 4. From this figure, it is clear that jacketing modified the behaviour of the existing structure so that all rehabilitated specimens behaved as "strong column - week beam" systems, in which beam contributions were the largest throughout the test. Comparing the differences between the specimens, the joint contribution of J1 increased remarkably with story drift and had already exceeded 50% of the total drift by the time 1% story drift was achieved. On the other hand, the joint component of Specimens J2 and J3 were approximately less than 25% and 30%, respectively. There were approximately no difference in the joint component between J3 with jacket on column and beam and J4 with jacket on column only.

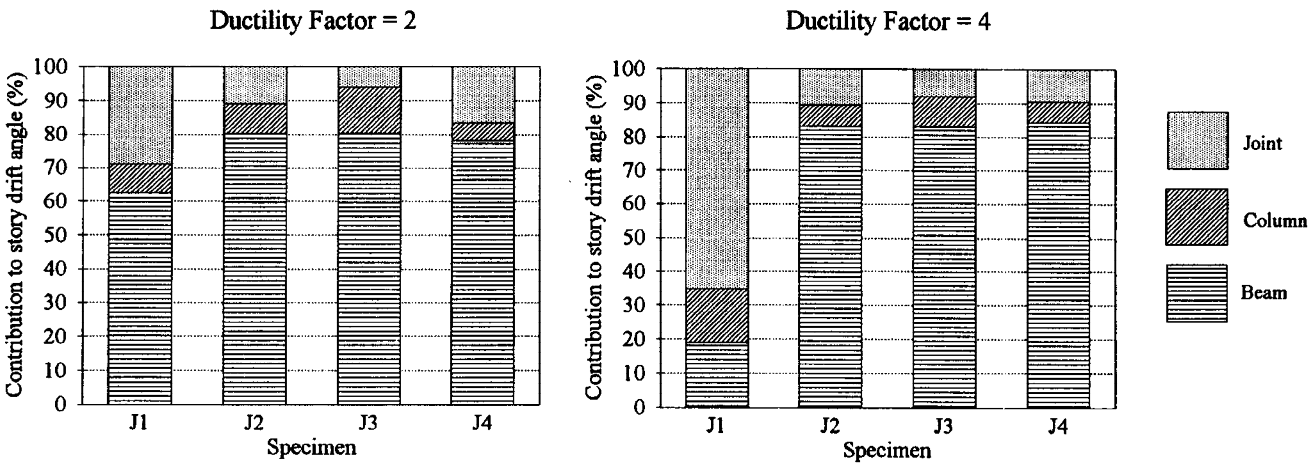


Fig. 9. Comparison between member contribution to drift angle.

Dissipated Energy. The energy dissipated during a loading cycle was computed as the area enclosed within the load-displacement curve. The cumulative energy dissipated was then calculated by summing the energies dissipated in consecutive loops throughout the test. The cumulative energy dissipation values are plotted in fig. 10 with the cycle number. The figure illustrates that specimens J2 and J3 exhibited considerably higher values for cumulative energy dissipation as compared to specimen J1. The cumulative energy dissipation values of specimen J4 are less than that of J2 and J3 due to the insufficient transverse reinforcement in the beam.

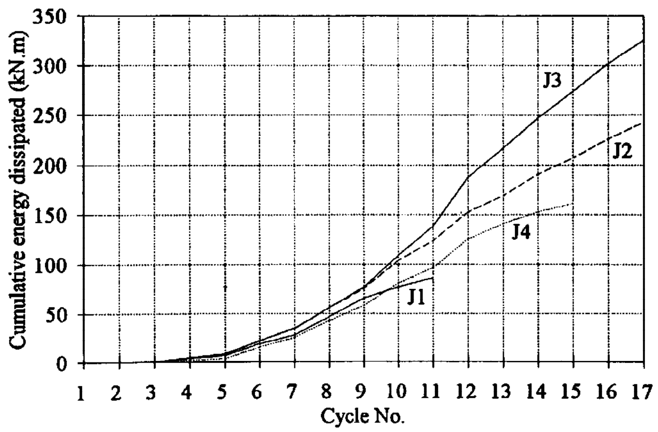


Fig. 10. Comparison of cumulative energy dissipated by the specimens.

CONCLUSIONS

From the observation and results of the experimental program, the following conclusions are arrived at:

1. An effective rehabilitation method was proposed and evaluated for deficient beam to column connections in existing reinforced concrete frames. Corrugated steel jackets provided the necessary shear strength and confinement to avoid distress of the joint region.
2. The confinement provided to the joints by the corrugated steel jackets increased the ductility of the specimen several times as compared to the ductility of the existing joints.
3. The steel jacket thickness and the grouted gap thickness designed to replace the deficiency of the transverse steel and the deficient confinement was found to provide sufficient shear strength and assure ductile behaviour.

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