Joint contribution to the deformation of RC beam-column sub-assemblies

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ABSTRACT: In this paper, the contribution of joint shear deformation to the overall storey-drift of reinforced concrete (RC) beam-column sub-assemblies is investigated experimentally. Two lightly reinforced beam-column sub-assemblies, one without any hoops inside the joint core and the other with hoops significantly less than that required by the incumbent seismic design codes, were tested under a constant axial compression and gradually increasing reversed cyclic displacements. Both specimens experienced severe damage in the joint due to excessive shear deformation of the joint core. Unlike in seismically designed ductile frames, joint shear deformation accounted for more than 50% of the overall storey-drift in the tested specimens. Comparison of the two test results showed that a small amount of hoops in the joint core, though not enough to satisfy seismic requirements, helps to confine the joint core and to inhibit the joint shear deformation to some extent.

1 INTRODUCTION

1.1 Background

Most finite element (FE) analysis programs model RC building frames with beam and column elements, which are connected to a node at the joint locations. This idealization is based on the assumption that the joint core is perfectly rigid. Representing the joint volume merely with a node yields a solution that does not account for the distortion of the joint core itself. As is well known, the overall deformation of a beam-column sub-assembly comes from the deformations of the beam and column and also from the distortion of the joint. Seismic design codes ensure that the joint cores of an RC building frame have sufficient amount of properly detailed shear reinforcement, so that the failure of the frame is triggered by the formation of a plastic hinge in one of the adjoining members. Quasi-static tests on such ductile joints have proved that the contribution of joint shear deformation to the overall storey-drift is much less than the contributions of the beam and column flexural deformations [Otani et al. 1985]. Although neglecting shear deformations of the joints in such a frame that is properly designed to meet seismic demands may be acceptable, there are cases where the shear deformation of joints cannot be overlooked. For example, buildings designed according to seismic design codes that prevailed a few decades ago may not satisfy the more stringent requirements of the incumbent seismic design codes. Due to sub-standard reinforcement detailing, joints in such building frames may not be as rigid as they were designed to be [Hakuto et al. 2000]. Moreover, RC building frames in moderate to low seismicity regions are designed to withstand the dead and live loads only. Hence, the resulting unconfined joints might also undergo excessive shear deformation when such buildings are subjected to any form of lateral loading.

1.2 Objective

The roles of transverse hoops inside a beam-column joint are identified differently by on going seismic design practices in different countries. The American standard [ACI-ASCE 91] states that the hoops are provided inside a joint mainly to resist the buckling of longitudinal column bars and to confine the concrete inside the joint core, thus increasing the compressive strength of the concrete strut which lonely dictates the shear resistance of a joint. On the other hand, the New Zealand standard [NZS 1995] decides the amount of joint hoops so that the required shear resistance is provided by the concrete and the hoops inside the joint core. Nevertheless in both standards, the allowable joint shear stress is interpreted as a function of concrete compressive strength only, and the amount of joint hoops is considered irrelevant once it exceeds the minimum recommended amount. Although such equations are not correct when the computation domain is the joint core only, they are acceptable for the seismic design of building frames, of which the joint is a non-critical part. However, the amount of joint hoops must be given due consideration in evaluating the strength of frames, of which the joint core is the weakest component.

In fact, if the design code recommendations were properly followed, the designed sub-assembly would first experience flexural failure of one of its members before the shear stress induced in the joint core is stretched to its limit. Hence, any further strengthening of the joint core has negligible effect on the overall capacity and deformability of the sub-assembly. In this pretext, to investigate the influence of surplus amount of joint hoops on the behaviour of the joint core is only of academic interest. Evaluating seismic performance of non-seismically designed connections may not appear to be a genuine idea, but it is an issue of prime importance and interest for researchers exploring the quality and quantity of retrofitting needed to strengthen such non-seismic connections. This paper tries to shed some light on the role of transverse hoops inside the joint core on the overall behaviour of lightly reinforced beam-column sub-assemblies. To satisfy the pre-requisites of the objective, it concentrates on a regime where the joint cores lack sufficient confinement and the behaviour of the sub-assembly is governed by the rigidity and strength of the joint core.

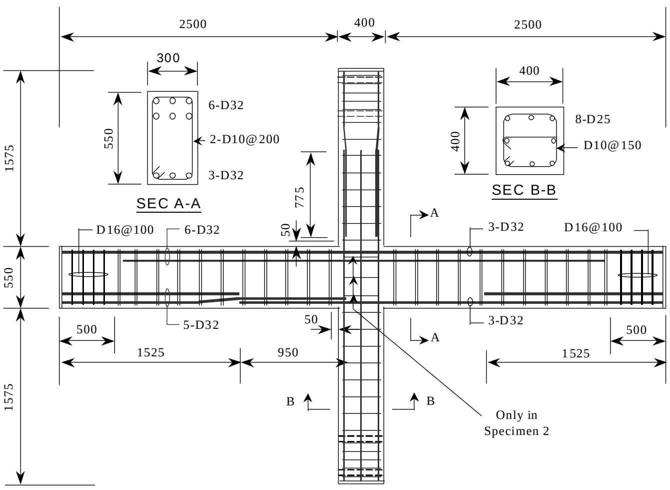

Figure 1. Reinforcement details of the specimens

2 EXPERIMENTAL FEATURES

2.1 *Specimen fabrication*

Figure 1 shows the geometrical properties and reinforcement details of the two full-scale lightly reinforced beam-column sub-assemblies which were tested under quasi-static cyclic loading. The two specimens had the same dimensions and similar rebar details except at the joints. The joint core of specimen 1 had no transverse hoops at all, whereas the transverse hoops of the column (three legs of 10 mm diameter bars with a spacing of 150 mm) were continued through the joint core in specimen 2, resulting in about 0.3% hoops inside the joint, which accounted for only about 40% and 25% of the hoops required by the ASCE [ACI-ASCE 1991] and the NZ [NZS 1995] standards, respectively. Making the matter worse, the column longitudinal bars were overlapped just above the joint. Note that such details are typical of RC connections of building frames in low and moderate seismicity regions [Beres et al. 1992] or old buildings designed according to the then existing seismic design codes [Hakuto et al. 2000].

For convenience, the specimens were cast in a horizontal plane. Analyzing the specimens based on the measured average strengths of the concrete and reinforcing bars used in the specimens revealed that the specimens were of the undesirable strong-beam weak-column type, and the joint was the weakest component in both sub-assemblies [Pan et al. 2002]. Hence, the specimens were expected to fail due to inadequate shear capacity of the joint core before any plastic hinge could be developed in the adjoining members. Furthermore, the joint cores were also expected to undergo large shear deformation, which may lead to severe damage in the joint surfaces.

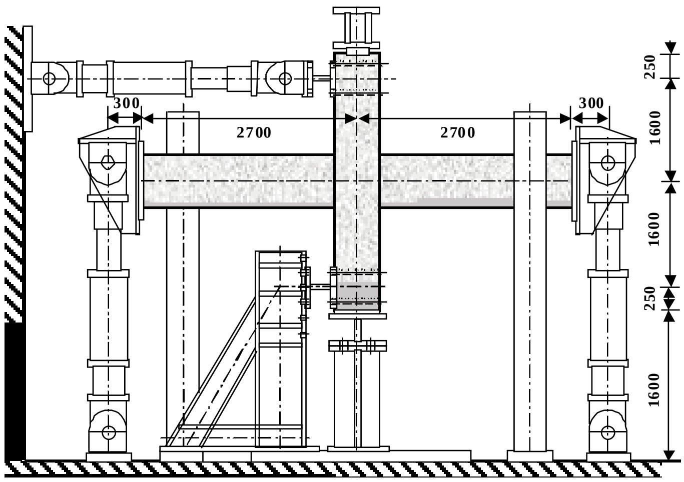

Figure 2. Schematic illustration of test set-up

2.2 *Test set-up and loading*

One of the specimens connected to the test rig is shown in Figure 2. The two end plates of the beam were pinned to vertical actuators, and the top end of the column was connected to a horizontal actuator through a universal pin joint. The bottom of the column was clamped to a pedestal against translation as well as rotation. Owing to the connection details, the effective height of the column was 3.2 m (distance between the centrelines of the supports at the top and the bottom) and the effective length of the beam was 6.0 m (distance between the centrelines of the actuators at the right and the left).

Axial compression equal to 10% of the axial capacity of the column cross-section was applied through two prestressing tendons connected to a steel H beam at the top of the specimen. Reversed cyclic displacements with gradually increasing amplitude were applied at the beam-tips through hydraulic actuators while restraining the movement of the column ends. Due to the symmetrical nature of the specimen, the displacements applied at the two ends at any instant were equal in magnitude but opposite in direction. A complete sequence of storey-drift cycles applied to the specimen is shown in Figure 3. Here, the storey-drift is the average rotation of the line joining the two beam-tips from the original beam-axis, and is equal to the summation of the two actuators' displacements divided by the effective length of the beam; i.e. 6.0 meters. As shown in the figure, the amplitude of the cyclic drift was increased gradually in steps until the specimens were damaged severely. Note that each step in the loading sequence had three repeated cycles corresponding to the same storey-drift, which allowed measurement of any strength degradation due to load repetition.

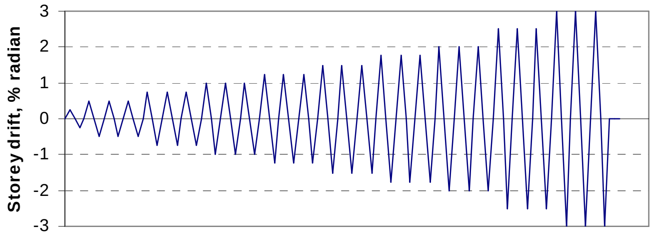

Figure 3. Applied displacement history

2.3 *Instrumentation and measurements*

LVDT transducers were used to measure the displacements at the beam-tips, and the support movements at the column-ends were measured with dial gauges. The vertical load at the beam-tips and the horizontal reaction at the column-top were measured with load cells. Most importantly, the shear deformation of the joint panel was measured with two pi-gauges attached diagonally across the joint panel.

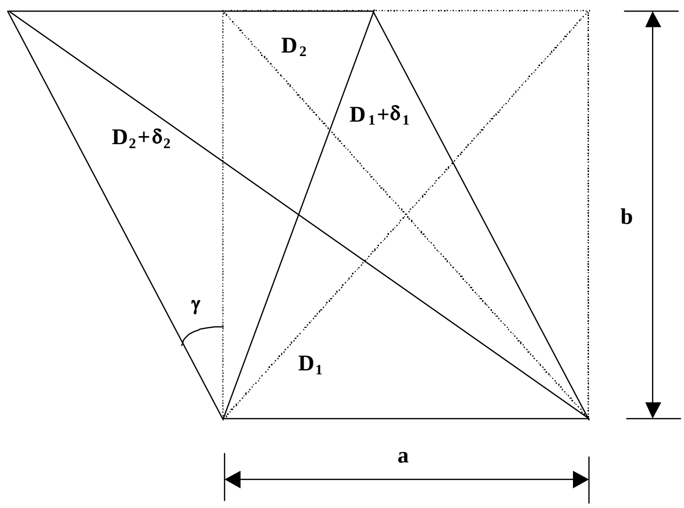

Figure 4. Computation of joint panel shear deformation

$$\gamma = \frac{(D_1 + \delta_1 + D_2 + \delta_2)(\delta_1 - \delta_2)}{4ab} \qquad (1)$$

As shown in Figure 4, these pi-gauges measured the elongation and/or shortening of the two diagonals of the joint panel. These measurements (δ_1 and δ_2) and the original dimensions of the joint panel (a, b, D_1 and D_2) were then used to derive joint panel shear deformation γ according to Equation 1. For data acquisition, all the gauges, transducers and load cells were connected to data loggers. Apart from these measurements, crack initiation and propagation were regularly monitored to track the overall damage process.

3 RESULTS AND DISCUSSIONS

3.1 *Cracks observed*

The first diagonal crack in the joint panel of specimen 1 could be seen after the 0.25% radian storey-drift cycles. On the other hand, the first diagonal crack could be observed in the joint panel of specimen 2 at the peak of the 0.5% radian storey-drift cycle. In both specimens, the first diagonal crack was followed by another orthogonal crack during the displacement reversal. This pair of diagonal cracks widened and some more distributed hairline cracks emerged in the joint panel during the later displacement cycles. Cracks in the joint panel of specimen 2 were more in number but smaller in width. A few small flexural cracks were also seen in the adjoining members, the columns having comparatively more cracks agreeing with the strong-beam weak-column status of the specimens.

Figure 5. Specimens at the end of the tests

Spalling cracks in the joint panel emerged at 1.5% and 2.0% radian storey-drift cycles in specimens 1 and 2, respectively. The concrete cover in the joint panel completely spalled out at 2.5% radian storey-drift in specimen 1 and at 3.5% radian storey-drift in specimen 2, after which the loadings were terminated. The physical condition of the two specimens after the test is depicted in Figure 5. As expected, both specimens were vulnerable to joint shear failure.

3.2 *Hysteresis loops*

As the specimens were symmetric, and equal and opposite displacements were applied at the two beam-tips, the load-displacement relationships observed at both loading points were similar. Because of the different amounts of reinforcement at the top and bottom of the beam, the maximum loads corresponding to the positive and negative displacements were not equal. Nevertheless, this did not affect the storey-shear force, which is a global parameter in equilibrium with the forces at the beam-tips. Figure 6 depicts the relationships between the storey-shear force and storey-drift for the two specimens. Here, the storey-shear force is the lateral force recorded by the load cell in the stationary actuator at the top of the column.

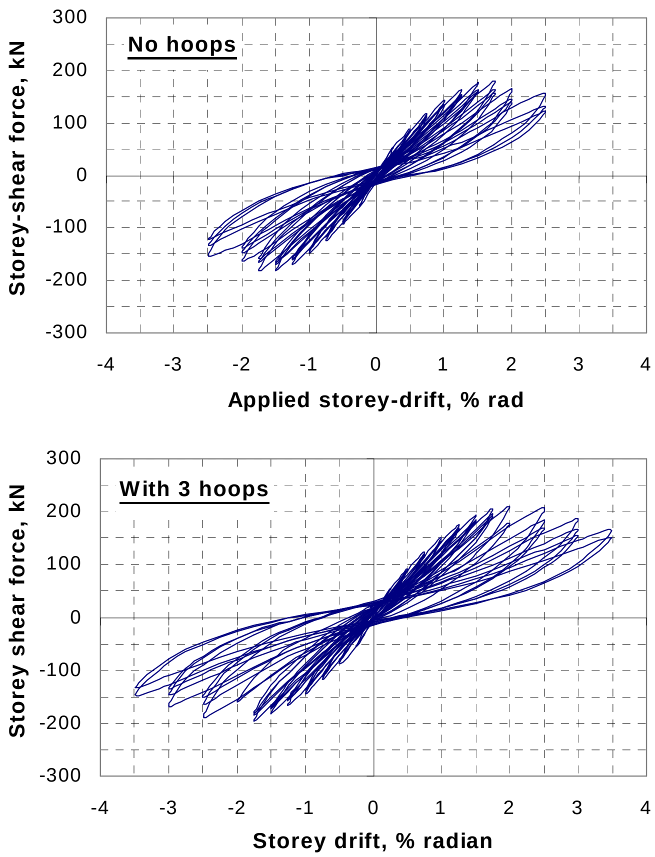

Figure 6. Storey-shear versus storey-drift relationships

Attributable to the stronger joint in specimen 2, the storey-shear capacity of specimen 2 (210 kN) was more than that of specimen 1 (179 kN). In both specimens, the maximum storey-shear force occurred when the applied displacement induced 1.75%-2.0% radian storey-drift. Both specimens exhibited significant pinching behaviour, and the drop in storey shear force between the first and the third cycles gradually increased in the post-peak region. Although specimen 1 lost its load carrying capacity after 2.5% storey-drift cycles, specimen 2 was still in loadable condition when the loading was terminated after 3.5% storey-drift cycles.

3.3 *Performance of the joint core*

Next, the shear deformation experienced by the joint panel was calculated using Equation 1. The interrelationship between the applied storey-drift and the induced joint panel shear strain in the two tests are plotted in Figure 7. Figure 7 also shows the contribution of joint shear deformation in the overall storey-drift at different loading stages for the two specimens. In these plots, average of the joint shear strains computed at the positive and negative peaks of the first cycle is used as the joint shear deformation. As can be observed, the rate of increase of joint shear strain abruptly increased after the applied storey-drift exceeded 1.75% radian.

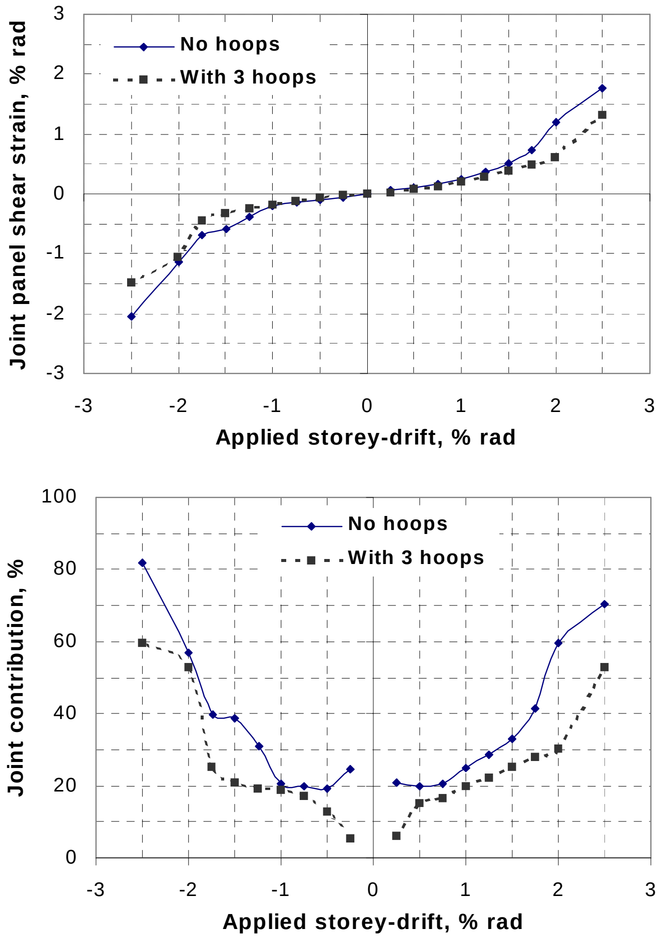

Figure 7. Variation of joint shear deformation with storey-drift

Note that the contribution of joint shear deformation to storey-drifts smaller than 1% radian did not exceed 20%, which is comparable to that in ductile frames. This is because the elastic deformations of the beam and the column consume a major share of the applied storey-drift in small displacement range. The three components; i.e. elastic flexural deformations of the beam and the column and the elastic shear deformation of the joint, increase proportionally until one of them goes into the inelastic range. In these tests, it is the joint shear deformation that first crossed its elastic limit after 1.5% storey-drift. At 1.75% storey-drift, the joint shear stress attained its maximum value, and any additional drift was consumed mostly by the inelastic shear deformation of the joint panel while the other two components remained almost unaltered. Hence, the joint contribution increased rapidly in the post-peak region.

3.4 Effect of joint hoops

Note that the three hoops inside the joint core of specimen 2 brought some remarkable improvements in its behaviour compared to that of specimen 1 which did not have any hoops in its joint core. Firstly, the capacity of the assembly increased by 17%. In ductile beam-column sub-assemblies where the beam or the column is the weakest component, a small change in the amount of joint hoops would not affect the overall capacity. Secondly, the deformability of the assembly increased considerably. Specimen 2 was loaded until 3.5% storey-drift (40% more than that of specimen 1) and it still seemed capable of withstanding further drift when the loading was terminated. The joint being the weakest component, the overall deformability of the assembly depended on the rigidity of the joint core, which was enhanced by the additional hoops. Again, the overall deformability would not have increased by the additional hoops had the beam or the column been the most critical component. In such a case, the critical member would yield before the joint shear deformation reached its limiting value, and the overall deformability of the assembly would depend on the flexural rigidity of the critical member. These two observations corroborate that participation of joint hoops in the shear resisting mechanism of the joint is significant in cases where low amounts of confinement are provided [Bonacci & Pantazopoulou 1993].

Lastly, the specimen with three hoops inside the joint core, though they were not enough to satisfy the seismic requirements, behaved more rigidly and significantly inhibited the shear deformation of the joint. As a representative figure, the three hoops (about 0.3% by volume) reduced the joint deformation by almost 0.4% at 2% storey-drift. Hence, it is foreseeable that additional hoops provided in the joint cores of ductile frames to satisfy seismic requirements would confine the joint contribution to a much smaller value the contribution of which can be overlooked in the analysis.

4 CONCLUSIONS

Two RC beam column sub-assemblies, one without any hoops the other with three three-legged 10 mm diameter hoops inside the joint core, were subjected to reversed cyclic displacements under a constant axial compression. As both specimens had a weaker joint than the adjoining members, they experienced a large joint shear deformation and were heading towards joint shear failure when the tests were terminated. Although the amount of the hoops in specimen 2 was only 40% and 25% of those required by the ASCE and NZ seismic design codes, they provided a non-negligible additional confinement to the joint core and enhanced the overall capacity and deformability of the sub-assembly by 17% and 40% respectively.

The contribution of the joint shear deformation to the overall storey-drift was more than 50% at the time of failure, which is significantly higher than that in seismically designed ductile beam-column sub-assemblies. One should hence be cautious in using dynamic analysis softwares, which idealize the joint as a node where the elements representing the adjoining members are interconnected. Such idealization is based on a rigid joint assumption, and is valid only when the joint deformation is negligible. Use of such tools to analyse RC frames with ductile joints where the contribution of joint shear deformation in the overall storey-drift is small may be acceptable for engineering purpose. But in no case, should such tools be used to analyse frames with lightly reinforced joints as the ones tested, because 50% contribution from joint panel shear deformation is too large to be overlooked. A separate element representing the joint core must be formulated and installed in such tools to enable them to accurately analyse such frames.

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