EXPERIMENTAL STUDY ON DYNAMIC RESPONSE OF GRAVITY-DESIGNED RC CONNECTIONS

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ABSTRACT: This paper reports an experimental program aiming to shed some light on the response of non-seismic RC beam-column joints to excitations of different frequencies. The RC connections tested were designed only for gravity loads, thus rendering the joint cores weaker than the adjoining members when subjected to a lateral load. Altogether, six tests were conducted on full-scale specimens, which were subjected to reversed cyclic displacements applied at different speeds varying from slow quasi-static loading to high-speed dynamic loading as fast as 20 Hz. Although all specimens expectedly suffered joint shear failure, the maximum joint shear stresses observed in the tested specimens, despite lacking transverse hoops inside the joint cores, were more than the horizontal shear stresses allowed in ductile RC joints with the same grade of concrete according to the existing seismic design codes. The damage patterns and failures of the specimens showed a better correlation with the residual storey shear stiffness rather than with the loss of storey shear strength during the repeated cycles. By analysing the test results, this paper also discusses how an inadvertent inertial force develops during high-speed displacement reversals.

Key words: Gravity-designed; beam-column joint; high-speed; inertial force; joint shear; residual stiffness

Introduction

Structures in low and moderate seismicity regions are designed to resist gravity loads comprising self-weight and superimposed weight. As seismic forces do not control the design, the beams and columns in an RC building in such regions intersect at joints that have no or very little transverse reinforcement. Even in seismic zones, RC buildings that were designed and built before seismic codes were fully matured may have a small amount of confining hoops in their joints, thereby failing to satisfy the stricter requirements of current design codes [1]. RC frames with such lightly reinforced joints, though being sound against gravity loads, are vulnerable when subjected to lateral loads. Although strong earthquakes are not likely to occur, buildings in non-seismic zones may in some cases be exposed to ground excitations of different frequencies that are induced by underground explosions, construction vibrations, long-distance earthquakes, etc. In order to assess the safety of buildings in non-seismic regions, the response and damage of such gravity-designed connections due to lateral cyclic loadings of static and dynamic natures should be well understood. The results of such studies also provide vital information and database that might finally lead to the development of new building design guidelines and recommendations on the nature and extent of strengthening needed to ensure the safety of existing buildings in such regions.

Although mechanisms related to seismic response of ductile RC connections with adequate shear reinforcement have been widely investigated and well documented [2-6], test data on gravity-designed joints are still scarce [1,7]. Indeed, most of the tests conducted are of quasi-static nature, and only a few dynamic tests [8] have been reported. The authors are unaware of any report on high-speed tests of any type of beam-column joints. Aiming to fill this void to some extent, this paper presents the results of an extensive experimental program consisting of static and dynamic tests on gravity-designed RC beam-column sub-assemblies. Some features inherent in the reinforcement details of the specimens and the loading protocol make these tests novel and special. Firstly, the joint cores of the specimens do not have any transverse reinforcement at all. Secondly, unlike in most tests reported in the existing literatures, the loading patterns adopted in the tests include reversed cyclic displacement at a frequency as high as 20 Hz. These unusual aspects add a new dimension to the importance of these test results, which provide information necessary to plan further studies on non-seismic RC frames.

Experimental Program

Specimen details and Test set-up

This paper discusses reversed cyclic loading tests of six RC beam-column sub-assemblies that are part of frames designed according to the British standard BS8110 [9]. The specimens were full-scale reproduction of a gravity-designed frame between the points of contra-flexure, which are assumed to be the mid-heights of columns in two successive storeys and the centre-points of beams in two adjacent bays. The six specimens tested were of two different types named C1 and C4 each with three specimens. The geometrical dimensions and reinforcement details of the C1 and C4 type specimens are illustrated in Figure 1 and are also listed in Table 1. Specimens of both types had similar overall dimensions (3.7 m high column and 5.4 m long beam), and the cross-section of the beam (300 mm width $\times$ 550 mm depth) was the same in all specimens. The beam in C1 type specimens had seven 32 mm diameter bars, five at the top (2.7% reinforcement ratio) and two at the bottom (1.1%), whereas C4 type specimens had six bars at the top (3.3%) and three bars at the bottom (1.6%) of the beam. C1 type specimens had columns with cross-section 350 $\times$ 500 mm, and two layers of four 25 mm diameter bars (2.4%) were laid parallel to the two longer sides. Similarly in C4 type specimens, the 400 $\times$ 400 mm column included eight 25 mm diameter bars (2.5%) arranged symmetrically along the perimeter. The stirrups in the beam comprised of four legs of 10 mm diameter bars spaced at 200 mm apart, and the ties in the column had three legs of 10 mm diameter bars with 150 mm spacing. Note that all specimens were without any vertical or lateral hoops inside the joint core. Moreover, the longitudinal reinforcing bars of the column were overlapped just above the joint, where the maximum moment occurs during a lateral loading. As can be seen in Figure 1, beam reinforcing bars at the bottom were discontinuous with the overlaps located close to the joint core, and the discontinuous bars were terminated with less than 100 mm penetration into the joint core.

All specimens were cast in horizontal position using formwork made of stiff steel panels to achieve the desired dimensions with minimal error. Standard compression test results of cylinders showed that the average compressive strength of concrete was 31.6 MPa for the C1 type specimens and 32.7 MPa for the C4 type specimens. Based on standard tension test results, the average yield strengths of the 32 mm, 25 mm and 10 mm diameter bars were 538 MPa, 537.6 MPa and 363.7 MPa respectively. Similarly, the average ultimate tensile strengths of these bars were 677.3 MPa, 675.3 MPa and 571.5 MPa respectively. A detailed strength analysis [10] revealed that these unfavourable details lead to specimens of undesirable strong-column weak-beam type and also vulnerable to joint shear failure. For beam-column joints with ductile details, the expressions to estimate nominal joint shear stress is $1.25\sqrt{f'_c}$ according to the American standards (referred to as ACI hereafter) [11], $0.2f'_c$ according to the New Zealand standards (referred to as NZ hereafter) [12] and $0.25f'_c$ according to the Japanese standards (referred to as AIJ hereafter) [13], where f'_c is the compressive strength of the concrete used (in MPa). Based on the measured concrete compressive strengths, these expressions yield 7.03 MPa, 6.33 MPa and 7.91 MPa for specimen type C1, and 7.15 MPa, 6.55 MPa and 8.19 MPa for specimen type C4. Note that the joint shear strengths of these specimens are supposed to be even lower as no transverse hoops existed inside the joint cores.

The specimens were connected to the test rig as schematically illustrated in Figure 2. The two end plates of the beam were pinned to vertical actuators, and the top end of the column was also connected to a horizontal actuator through a universal-pin joint. The bottom of the column was clamped to the rig against translation as well as rotation. The set-up adopted in these tests was hence different from the usual test set-up [1,3-7], which used pin joints for all four supports. A steel H beam was placed on the column-top and was clamped with the strong floor through two prestressing tendons used to apply axial compression. Note that 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).

Loading patterns

All specimens were subjected to an axial compression at the column-top. Although the axial load was intended to remain constant throughout the test, it was found to vary between 11-13% of the axial capacity of the column cross-section. Note that this axial load is lower than that acting on columns in buildings, and the results may hence be slightly conservative. The specimens were subjected to reversed cyclic displacements with gradually increasing amplitudes applied at different speeds. In quasi-static tests on a beam-column sub-assembly, cyclic loadings can either be applied at the column-top [1,3,4,6] or at the beam-tips (this does not include P-delta effects) [5,7] while keeping the other supports stationary. To facilitate the application of a constant axial compression, the cyclic loadings in these tests were applied at

the beam-tips. Due to the symmetrical nature of the specimens, the displacements applied at the two beam-tips at any instant were equal in amount but opposite in direction. Based on the loading protocols, the tests were divided into three categories, namely pseudo-dynamic (PD) tests, normal-dynamic (ND) or constant frequency tests, and high-dynamic (HD) or varying frequency tests, respectively. Two specimens (one each of types C1 and C4) were subjected to each of these three loading patterns. The name of each specimen consists of the specimen type followed by the initials for the loading pattern used. For example, C4PD refers to the specimen of C4 type subjected to pseudo-dynamic loading. The six specimens were thus named as C1PD, C1ND, C1HD, C4PD, C4ND, and C4HD, respectively.

Complete sequence of the storey drift cycles applied to the specimens during the two pseudo-dynamic tests is shown in Figure 3. Here, the storey drift is computed by the summation of the two actuator displacements divided by the distance between the actuators; i.e. 6.0 meters. For example, 0.5% storey drift corresponds to 15 mm displacement of the two actuators at the beam-tips, but in opposite directions. As shown in the figure, the amplitude of the storey drift cycles was increased gradually in steps from 0.25% to 2.0% with a 0.25% increment in each step. After reaching 2.0%, the increment was increased to 0.5% until the specimen failed. The first cycle corresponding to 0.25% storey drift was applied once only, and each cycle after that was repeated three times to observe strength degradation due to the load repetition, if any.

In the constant frequency tests, the two ND specimens were subjected to moderate-speed reversed cyclic displacements applied at two cycles per second; i.e. 2 Hz. Maintaining the cyclic loading frequency constant, the magnitude of the displacement cycles was gradually increased in the same manner as in the pseudo-dynamic tests. In the varying frequency tests, the two HD specimens were subjected to displacement cycles with frequency gradually changing from 20 cycles per second (20 Hz) to 2 cycles per second (2 Hz). Table 2 lists the amplitude, number of cycles and cyclic frequency of the displacement cycles applied to the HD specimens. The loading history in the varying frequency tests started with 10 cycles each of ± 2 , ± 5 and ± 10 mm amplitude, and then was followed by the loading sequence of the pseudo-dynamic tests from the $\pm 0.5\%$ storey drift cycles. Note that the displacement cycles in the varying frequency tests were applied at the maximum frequencies possible with the actuators without exceeding the oil supply limit. Consequently, starting from the ± 2 mm cycles at 20 Hz, the cyclic frequency gradually decreased with an increase in the displacement amplitude, and the cycles corresponding to 3.0% and larger storey drift could be applied only at 2 Hz, similar to those in the constant frequency tests. In fact, decision on the combinations of displacement amplitudes and cyclic frequencies was also inspired by the finding that the higher frequency excitations generate higher order vibration modes, for which the displacement responses are much smaller than those for the fundamental modes generated by resonant responses [14].

Objective of the Tests

The tests discussed in this paper are a part of a research project [10] conducted by the Protective Technology Research Centre (PTRC), Nanyang Technological University (NTU), Singapore, in collaboration with the Defence Science and Technology Agency (DSTA), Singapore, and the National Center for Research on Earthquake Engineering (NCREE), Taiwan. The objective of the project was to assess the performance of gravity-designed building frames against explosion-induced ground shocks. Therefore, the beam-column subassemblies tested were taken from a low-rise industrial building in Singapore to represent the practice in low to moderate seismicity region. As the design of these buildings was controlled by gravity loads [9], the specimen details are noticeably different from those adopted in seismic regions. Given the details, it may appear obvious that the specimens would undergo joint shear failure when subjected to quasi-static lateral loads, but the main intent of the project was not to check the failure mode but to investigate the response of such frames under high-frequency ground shocks. Moreover, to investigate the effect of explosion-induced ground shocks, the specimens were subjected to displacement cycles at unusually high speed. Note that explosions induce high-frequency and large-amplitude shocks, which will force the structures to respond in high-frequency modes during the excitation period [14]. Hence, the force-vibration response will include high-frequency and small-amplitude oscillations whereas the free-vibration response, as usual, will be dominated by large-amplitude oscillations at natural resonant frequency. The loading pattern was decided to cover both of these response phases.

Instrumentation and data acquisition

The locations of external instruments used to measure the response of the specimens are schematically illustrated in Figure 4a. The applied displacements and the resisting forces at the loading

points were measured with LVDTs and load cells inherent in the actuators. Similarly, a load cell and an LVDT in the actuator at the column-top measured respectively the lateral force and the support movement, if any. In a normal test set-up with all pinned supports, the force at the column-top would not need to be measured explicitly as it could be derived from static equilibrium of the loads measured at the beam-tips. However, the direct measurement of the lateral reaction at the column-top became mandatory in the adopted set-up to make the system statically determinate, because the fixity of the bottom support would induce an additional unknown parameter in moment. To take into account the rigid body rotation, movement of the bottom support was also recorded through a dial-gauge. The axial compression force was monitored using two external load cells connected to the two prestressed tendons.

Note that two pi-gauges were attached diagonally across the joint surface in order to record the elongation or shortening of the diagonals. As shown in Figure 4b, these pi-gauge readings (δ_1 and δ_2) combined with the original width (a), height (b) and diagonal length (D) of the joint panel give the shear-deformation γ undergone by the joint. In addition, tiltmeters were attached along the four sides of the joint panel to measure the rotation of the adjoining members. Though not shown in the figure, several strain gauges were attached to the longitudinal and transverse reinforcing bars near the joint core to monitor if plastic hinges would develop in the beams or columns. For data acquisition, all of the abovementioned measuring devices were connected to normal data loggers in the pseudo-dynamic tests and to special high-speed data loggers in the dynamic tests. Precaution was taken in selecting and using the loading devices, response measuring gauges and data recording instruments; especially for the high-speed dynamic tests [15]. In the pseudo-dynamic tests, readings were taken with predefined displacement intervals whereas in the high-speed tests, the sampling frequency was adjusted to 200 Hz; i.e. 200 readings with 5-millisecond intervals in one second. Apart from these measurements, in order to observe the overall damage process, crack initiation and propagation were monitored closely while maintaining the continuity of the displacement cycles to fulfil the test objectives.

Test Results

Pseudo-dynamic (PD) tests

In the two PD tests, the cyclic displacements were temporarily stopped at the opposite peaks of the first and the third cycles of each loading step to observe and mark the cracks. Diagonal cracks first appeared in the joint panel surfaces before any crack could be noticed in the beams and the columns. In both specimens, the first hairline crack could be noticed at the peak of the 0.25% storey drift cycle. The first pair of diagonal cracks in the joint panel was clearly visible at the peaks of the first cycle corresponding to 0.5% storey drift. Some more cracks developed randomly in the joint panel surfaces on further loading, but these new distributed cracks were much thinner compared to the first pair of diagonal cracks, which opened and closed alternately following the direction of the cyclic loading. A few uniformly distributed flexural cracks also emerged in the adjoining members. During the larger displacement cycles, the upper column-joint interface opened and the concrete cover spalled out from the joint panel thereby exposing the reinforcing bars. The joint panel deformed significantly and was damaged severely, which prompted the termination of the C1PD test after 3.0% storey drift cycles and the C4PD test after 2.5% storey drift cycles. Figure 5 shows the physical condition of the joint panel of the two specimens at the end of the tests, which clearly depicts the significantly deteriorated joint panels. The maximum strains recorded in the main bars of the beams (C1PD~ $0.4\epsilon_y$, C4PD~ $0.3\epsilon_y$) and the columns (C1PD~ $0.9\epsilon_y$, C4PD~ $0.6\epsilon_y$) were less than the yielding strain, corroborating that the specimens were heading towards joint shear failure.

Figures 6a and 6b show the relationships between the storey shear force and the average storey drift of the two PD specimens. Here, the storey shear force is the lateral reaction recorded by the load cell at the column-top, and the average storey drift is the angle made by the line joining the beam-tips with respect to the original beam axis. The hysteresis loops of both specimens show severe pinching, and the storey shear force started decreasing immediately after attaining its peak value. The maximum storey shear force occurred when the applied displacements induced 1.75% storey drift in C1PD and 1.5% storey drift in C4PD. Owing to the larger joint cross-section in C1 type specimens, the maximum storey shear force of C1PD (224.74 kN) is higher than that of C4PD (180.54 kN). As shown in Figure 6c, the difference between the storey shear forces read at the peaks of the first and the third cycle is less than 10% before the maximum storey shear force is reached, and it increases rapidly in the post-peak region. When the visible damage prompted the termination of the tests after applying 2.5% and 3.0% storey drift cycles, the difference between the shear force readings at the peaks of the 1st and the 3rd cycle had just exceeded 20% in both tests.

Figure 6d plots the shear stiffness degradation patterns of the two PD specimens with respect to the applied storey drift. Here, the residual stiffness is presented in terms of percentage of the initial stiffness, a representative value of which is obtained from the storey shear force readings taken at 0.125% and -0.125% storey drifts of the first cycle. As the first hairline crack appeared at 0.25% storey drift, the computed stiffness at 0.125% storey drift fairly represents the initial elastic stiffness of the specimens. The stiffness corresponding to a storey drift is calculated as the average of the secant stiffness at the opposite peaks of the first cycle corresponding to that storey drift. The trend of stiffness degradation was very similar in both specimens, and gradual stiffness degradation could be observed throughout the applied storey drift range. Correlating the observed physical condition of the specimens with the stiffness degradation patterns, it can be approximated that 80% loss in the structural elastic stiffness corresponded to a major damage of the tested specimens. In fact, both tests were terminated when the residual stiffness was about 20% of the initial representative stiffness.

As both specimens were vulnerable to joint shear failure before the formation of a plastic hinge in any of the adjoining members, the joint panel shear deformation must be large and the maximum storey shear force must have occurred when the shear stress in the joint panel was equal to its capacity. For further verification, Figures 6e and 6f illustrate the shear stress-strain envelopes of the joint panels of the specimens C1PD and C4PD respectively. To satisfy equilibrium, the horizontal joint shear force must be equal to the difference between the sum of bar forces on the two sides of the joint and the column shear force. The bar forces could be obtained by dividing the beam moments by the arm length (assumed constant and equal to 7/8 of the beam depth), and the sum of the beam moments could again be approximated as the column shear force multiplied by the specimen height. The joint shear stress is then obtained by dividing thus derived horizontal joint shear force by the effective joint area where the width of the joint core is taken as an average of the beam and column widths [11]. Note that a constant arm length corresponds to perfect bond throughout the tests, and the inclusion of bond deterioration in the computation increases the joint shear stress corresponding to a storey shear force [16]. The maximum joint shear strain at 2.5% storey drift was 0.0124 radian (50% contribution) in C1PD and 0.0176 radian (70% contribution) in C4PD, which are much higher than those reported in literatures for frames with ductile and non-ductile details [17,18]. The nominal joint shear stresses recommended by the ACI [11], the NZ [12] and the AIJ [13] standards are also shown in Figures 6e and 6f. As can be seen, the observed joint shear strength of C1PD (7.81 MPa at 0.0066 radian) is almost equal to the AIJ nominal stress which is larger than the values recommended by the other two codes, whereas that of C4PD (7.29 MPa at 0.0058 radian) is very close to the ACI recommendation which falls between the nominal joint stresses recommended by the AIJ and NZ standards.

Constant frequency (ND) tests

The crack initiation and propagation in the ND tests could not be observed, as the applied loading was fast (2 cycles per second) and uninterrupted. The first damage inspection was performed when the cyclic loading was stopped after completing the 2.5% storey drift cycles. This drift ratio was selected because it was the instant when the specimens during pseudo-dynamic tests incurred heavy damage leading to the termination of the tests. After the 2.5% storey drift cycles, both specimens had many cracks in the joint panel surfaces, and spalling of cover concrete had just begun at some locations. As the specimens did not show any sign of failure at this stage, larger drift cycles were applied until failure, but the damage on the specimens was inspected after three displacement cycles corresponding to a storey drift. Finally, the tests were terminated after the 4.0% storey drift cycles had been applied, when the reinforcing bars inside the joint panels were fully exposed, as shown in Figure 7. Although the strains in the reinforcing bars were well below the yielding strain (C1ND: beam~0.5 ϵ_y column~0.7 ϵ_y , C4ND: beam~0.3 ϵ_y column~0.4 ϵ_y) and plastic hinges had not been formed in the adjoining members at that stage, the joint had been severely damaged.

Cyclic relationships between the storey shear force and the storey drift of the C1ND and C4ND specimens are shown in Figures 8a and 8b, respectively. Note that the curves are not as smooth as those of the PD specimens because of the very high data sampling frequency used to capture the response during the high-speed loadings. The maximum storey shear force occurred during the 1.75% storey drift cycle in C1ND (258.52 kN) and 2.0% storey drift cycle in C4ND (206.49 kN). Figure 8c shows the variation of the difference between the storey shear forces recorded at the peaks of the 1st and the 3rd cycles with the applied storey drift in the two ND tests. Note that the reduction of the storey shear force during the three repeated cycles increased until 2.5%-3.0% storey drift, after which it decreased unexpectedly. The maximum cyclic shear degradation was about 15% in C4ND and slightly more than 20% in C1ND. Nevertheless, the specimens had not undergone enough damage to suggest a failure at that

stage, and the cyclic shear degradation before the termination of the tests was much less, thereby making it difficult to correlate the loss of storey shear force in three cycles with the failure.

Stiffness degradation patterns of the two ND specimens are plotted in Figure 8d. As in the PD tests, the residual stiffness decreased gradually with an increase in the applied storey drift. When the specimens were severely damaged and the tests were terminated after completing the 4.0% storey drift cycles, the residual stiffness was less than 20% of the representative initial stiffness corresponding to 0.125% storey drift. The tested specimens had lost about 80% of their initial stiffness when they started suffering major damage in the joint panels after the 3.0% storey drift cycles had been applied. Next, the envelopes of the cyclic relationships between the horizontal shear stress at the centre of the joint and the shear strain that the joint panel had undergone are shown in Figures 8e and 8f for the specimens C1ND and C4ND, respectively. Note that the data used to plot these curves correspond to storey drifts up to 2.5%, because the pi-gauges attached on the joint panel surface were either disrupted or dislodged by the dropping pieces of concrete, and the readings could not be taken during the larger drift cycles. The maximum joint panel shear strain developed during the 2.5% storey drift cycles was 0.0073 radian (29% contribution) in C1ND and 0.0082 radian (33% contribution) in C4ND. The nominal horizontal joint shear stresses estimated according to the design codes [11-13] for ductile joints with the same grade concrete are also illustrated in the figures. As can be observed, the maximum horizontal joint shear stresses computed without considering any bond deterioration (8.99 MPa at 0.0041 radian in C1ND and 8.33 MPa at 0.0058 radian in C4ND) are not less than the nominal stresses recommended by any of the codes.

Varying frequency (HD) tests

In order to explore the effect of the very high frequency but small amplitude cycles, the cyclic loading in the varying frequency tests was stopped after each loading step (i.e. after applying the displacement cycles corresponding to a constant storey drift) until the appearance of the first crack. The initial 30 high-frequency cycles (amplitude less than 10 mm and frequency more than 10 Hz) did not inflict any cracks on the specimens. In fact, the first pair of diagonal cracks could be seen after the 0.5% storey drift cycles in C4HD and after 0.75% storey drift cycles in C1HD. Thereafter, the loading was stopped only when the 2.5% storey drift cycles had been applied. Hence, emergence of new cracks and the growth of the first diagonal cracks could not be tracked during the process. After being subjected to the 2.5% storey drift cycles, the specimens had several cracks in the joint panel; especially those in C4HD were much wider. As shown in Figure 9, the specimen C4HD lost all the concrete cover before the test was terminated after the 3.0% storey drift cycles, whereas the specimen C1HD could be loaded until 4.0% storey drift cycles before suffering a similar level of damage. Note that the maximum strains measured in the longitudinal reinforcing bars in beam (C1HD~ $0.9\epsilon_y$, C4HD~ $0.65\epsilon_y$) and column (C1HD~ ϵ_y , C4HD~ $0.8\epsilon_y$) outside the joint panel were close to the yielding strain, unlike in the PD and ND series.

The results of the two HD tests are compiled in Figure 10. Figures 10a and 10b show the hysteresis loops between the storey shear force and the applied storey drift in the specimens C1HD and C4HD, respectively. As in the constant frequency tests, the high data-sampling rate impaired the smoothness of the curves. The loops are significantly pinched and show rapid reduction of storey shear force in the post-peak region. The maximum storey shear force in the specimens C1HD (251.6 kN) and C4HD (229.3 kN) occurred during the first cycle inducing 1.75% storey drift. Figures 10c and 10d plot respectively the variations of the storey shear force lost in the three repeated cycles and the residual shear stiffness with respect to the applied storey drift. Unlike in the previous tests, the variation of the difference between storey shear force readings at the peaks of the 1st and the 3rd cycles with the storey drift was significantly different in the two specimens. In fact, the cyclic degradation of storey shear force varied randomly, showing no correlation with the applied storey drift level. On the other hand, the stiffness degradation pattern qualitatively remained the same as in the previous tests despite the difference in the loading speed. Note that a higher initial stiffness resulted in relatively lower percentile residual stiffness at larger storey drifts. The residual stiffness in both tests was less than 20% when a visibly severe damage forced the termination of the tests.

The shear stress-strain envelopes of the joint panels in the specimens C1HD and C4HD are plotted in Figures 10e and 10f, respectively. As in the constant frequency tests, the joint panel shear strain could not be measured beyond the 2.5% storey drift cycles, and the maximum shear strain observed was equal to 0.0124 radian (50% contribution) in C1HD and 0.0095 radian (38% contribution) in C4HD. As depicted in Figures 10e and 10f, comparison of the maximum joint shear stresses induced in the two specimens (8.75 MPa at 0.006 radian in C1HD and 9.25 MPa at 0.0062 radian in C4HD) with the

nominal stresses recommended by different design codes [11-13] shows that both specimens could withstand a joint shear stress larger than those allowed by the design codes.

Discussion

As the joint was the weakest component of the tested sub-assemblies [10], damage was mostly concentrated in the joint panel in all tests. As expected, the joint panels deformed significantly in all tests, and all specimens experienced joint shear failure regardless of the loading speed. However, the maximum shear stresses developed in the joint panels in all specimens were higher than the nominal joint shear stresses recommended by the ACI [11] and the NZ [12] standards, and those in all except C4PD exceeded the AIJ [13] recommended nominal stress for ductile joints with the same grade concrete. As the joint shear stress would even increase if the bond deterioration was taken into account [16], the aforementioned observation indicates that the expressions provided in the existing codes to compute the nominal joint shear stress of ductile joints may also be used to get a conservative approximation of shear capacity of gravity-designed joints.

In the pseudo-dynamic tests, 20% drop of storey shear force in three repeated cycles was interpreted as the failure of the tested specimens. In contrast, the cyclic degradation of shear force during the high-speed tests did not show any consistent correlation with the physical condition of the specimens, thereby making it impossible to calibrate damage during high-speed loading using the loss of storey shear force in three repeated cycles. On the other hand, shear stiffness gradually decreased with an increase in the storey drift during all tests regardless of the loading speed, indicating that residual shear stiffness might better track the damage caused by cyclic loading of any speed. Based on the observed physical condition of the specimens while keeping in mind the uncertainties involved in the estimation of the initial stiffness, it appeared that 80% loss of the shear stiffness could be approximately categorized as a major damage.

Effect of Inertia Forces

As the specimens were symmetric and equal and opposite displacements were applied at the beam-tips, the load-displacement relationships at the two loading points were identical in all tests. The cyclic load versus displacement curves at one of the loading actuators of a pseudo-dynamic (C4PD), a constant frequency (C4ND) and a varying frequency (C4HD) tests are shown in Figures 11a, 11b and 11c, respectively. In all of these curves, the maximum actuator force recorded in the two opposite directions are not equal due primarily to the different amounts of reinforcement at the top and bottom of the beam. Interestingly, the curves in the higher-speed tests (C4ND and C4HD) are found to suddenly unload at the positive and negative peaks of each displacement cycle. The extent of unloading in C4ND becomes more prominent during the larger displacement cycles, whereas it is prominent even during the smaller displacement cycles in C4HD. A scrutiny of the results [15] revealed that this behaviour is mainly attributable to the development of a large acceleration and thus an inertial force in the direction opposite to that of the displacement being applied.

As shown in Figure 12, the displacement reversal at the positive peak of each high-speed cycle induces a sudden change of velocity from a positive to a negative value. Correspondingly, a negative spike will be formed in the acceleration history. Nevertheless, due to mechanical limitations, the loading actuators cannot switch sharply their movement from/to outward to/from inward direction, and needs some time in doing so. Consequently, there appears a smooth transition phase around the peaks of each displacement cycle. The maximum displacement that could be applied is thus slightly less than the intended magnitude. As the transition phase is very short, the induced negative acceleration becomes large; thus generating a significant inertial force. The magnitude of thus developed inertial force depends on the loading speed in mm/sec and not on the cyclic frequency in Hz. As the constant cyclic frequency makes the loading speed (mm/sec) proportional to the amplitude, the inertial force gradually increases with the amplitude of the displacement cycles in the ND specimens. In contrast, a combination of decreasing frequency and increasing amplitude keeps the loading speed (mm/sec) within a fixed range in the HD specimens, thereby resulting in a significant inertial force that does not change much with the amplitude of the applied displacement cycles.

In fact, the inertial force affects only the load cell readings in the high-speed loading actuators, and the load cell readings at other stationary locations are not influenced at all. In the set-up and loading pattern adopted in these tests, inertia force could not be generated at the column-top as it was restrained against lateral movements. Hence, the load cell reading at the column-top correctly represented the storey shear force. That is why the storey shear force versus storey drift plots discussed earlier did not show the unloading dips at the peaks of the displacement cycles. Had the storey shear forces been derived from the

readings of the two loading actuators instead of measuring directly at the column-top, the influence of the inertial force would have been prominent in those curves. Also note that the same would have happened if the cyclic displacements were applied at the column-top instead of at the beam-tips.

Conclusions and Recommendations

Cyclic loading tests of six full-scale RC beam-column sub-assemblies were successfully conducted. Gradually increasing displacement cycles were applied at different speeds to the specimens, which were designed only for gravity loads and hence had no hoops inside the joint cores. Based on the results documented in this paper, the following conclusions can be drawn:

1. When gravity-designed RC frames with joint as the weakest component are subjected to lateral actions, they experience severe damage in the joint panels and ultimately suffer joint shear failure before the formation of a plastic hinge in the adjoining members.
2. The joint panel in a gravity-designed RC frame undergoes much larger shear deformation. It is hence recommended that the joint panel shear deformation, which is commonly overlooked in the analysis of ductile frames, be properly taken into account while dealing with the kinematics in the analysis of gravity-designed RC frames.
3. The shear strength of gravity-designed joints subjected to quasi-static loading may be conservatively approximated by the expressions provided in the existing seismic design codes to estimate the nominal shear stress in ductile joints. However, the joint shear strength was found to increase with an increase in the loading speed, and the code expressions do not take into account the loading speed.
4. Although 20% loss of shear strength in three repeated cycles may reasonably be interpreted as failure of gravity-designed frames due to quasi-static cyclic loading, physical condition of joints subjected to high-speed loading did not correlate to the degradation of shear strength. Instead, the test results showed that 80% loss of the initial shear stiffness corresponds to severe damage of such frames regardless of the loading speed. Although such interpretations may be applicable for laboratory tests with controlled and regular loading, a better definition of failure is needed to be applicable to real structures subjected to random loading.
5. A large acceleration and consequently an inertial force are unavoidably induced at the loading points during high-speed displacement reversals. The direction of thus generated acceleration and inertial force is opposite to that of the displacement being applied and their magnitudes are proportional to the loading speed.
6. In order to measure the correct storey shear force devoid of any inertial effect, it is necessary to apply the high-speed cyclic displacements at the beam-tips and to directly record the storey shear force through a load cell at the stationary column-top. Even if all supports are pinned and it is possible to derive the storey shear force from the beam-tips load cell readings, it is strongly recommended that an additional measurement be provided at the column-top to keep the storey shear force from being contaminated by the inertial effect.

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Figure 1 Geometrical features and reinforcement details of the specimens
Figure 2 Test set-up
Figure 3 Cyclic loading history followed in the PD and ND tests
Figure 4 External instrumentation plan and computation of joint panel shear strain
Figure 5 Specimens at the end of the pseudo-dynamic (PD) tests
Figure 6 Pseudo-dynamic (PD) test results
Figure 7 Specimens at the end of the constant frequency (ND) tests
Figure 8 Constant frequency (ND) test results
Figure 9 Specimens at the end of the varying frequency (HD) tests
Figure 10 Varying frequency (HD) test results
Figure 11 Typical load-displacement relationships at one of the loading points in the three test series
Figure 12 Negative acceleration generated during high-speed displacement reversals

Table 1 Specimens dimensions and details

	Type C1	Type C4
Column height, mm	3700	3700
Beam length, mm	5400	5400
Column cross-section	350 mm × 500 mm	400 mm × 400 mm
Beam cross-section	300 mm × 550 mm	300 mm × 550 mm
Concrete cover, mm	Column (35-50), Beam (20-35)	Column (35-50), Beam (20-35)
Column main bars	8 × 25 mm bars (2.38%)	8 × 25 mm bars (2.45%)
Column ties	2 × 10 mm bars @ 150 mm c/c	3 × 10 mm bars @ 150 mm c/c
Beam bars (top)	5 × 32 mm bars (2.74%)	6 × 32 mm bars (3.28%)
Beam bars (bottom)	2 × 32 mm bars (1.05%)	3 × 32 mm bars (1.57%)
Beam stirrups	4 × 10 mm bars @ 200 mm c/c	4 × 10 mm bars @ 200 mm c/c

Table 2 Loading sequence for HD test series

Displacement amplitude, mm	Storey drift angle, % rad	No. of cycles	Frequency, Hz
±2 mm	0.067%	10	20 Hz
±5 mm	0.17%	10	15 Hz
±10 mm	0.33%	10	10 Hz
±15 mm	0.5%	3	10 Hz
±22.5 mm	0.75%	3	8 Hz
±30 mm	1.0%	3	7 Hz
±37.5 mm	1.25%	3	6 Hz
±45 mm	1.5%	3	5 Hz
±52.5 mm	1.75%	3	4 Hz
±60 mm	2.0%	3	4 Hz
±75 mm	2.5%	3	3 Hz
±90 mm	3.0%	3	2 Hz
±105 mm	3.5%	3	2 Hz
±120 mm	4.0%	3	2 Hz

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

Figure 7

Figure 9

Figure 11