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Full-scale impact tests of columns for rocking steel braced frames

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ABSTRACT

The article introduces a test program that was conducted to investigate force and acceleration demands in beams and columns caused by column impacts at the base of rocking steel braced frames with self-centring capacity supplied solely by gravity loading. The tests were performed on a full-scale column of a rocking braced frame of a prototype two-storey building. A roof beam and a floor beam supporting representative gravity loads were connected to the test column. During the tests, the column was dropped from an initial uplifted position and the column vertical displacement response was recorded before and after the impact, together with vertical accelerations and member forces induced in the columns and the beams. The initial uplift was gradually increased from 25 to 145 mm, the latter value corresponding to 2.2% drift in the reference prototype building. The tests showed a significant increase in column forces and beam forces due to column uplift and impact. Following the impacts, up to 5 rebounds triggered peaks of vertical acceleration up to 10g in the floor beam and 5g in the roof beam. The axial force demands in compression were increased just after the impact up to a factor of 8.9 times the static value. Bending moment and shear in beams were increased up to a factor of 8 and 6.2, respectively. No major structural damage was noticed after the test, whether in the frame members or in the foundation.

Keywords: Steel braced frame; Rocking; Column impact; Full-scale test;

INTRODUCTION

Controlled rocking braced steel frames have been introduced in construction to enhance the seismic performance of building structures [1-2]. In this system, the columns of the braced frame are allowed to lift from the foundations during a severe earthquake. For these frames, self-centring capacity can be achieved either by the gravity loads supported by the frame [3], vertical post-tensioned tendons [4], pre-compressed base springs [5], or a combination thereof. Energy dissipation mechanisms are generally introduced at the base of the columns to control drifts of the building. Past numerical and experimental studies have shown that the system can significantly reduce seismic induced member forces, while exhibiting uniform and limited displacements over the building height [6]. More importantly, the frames can withstand severe earthquake demand without any structural damage. The system has already been implemented in new structures in New Zealand and California [7-8].

In most previous research, rocking braced frames were physically separated from the gravity load resisting system of the building to avoid imposing vertical displacements on the adjacent building structure. Such isolated frames support negligible gravity loads and must rely on tendons and/or springs to develop the desired self-centring response. For low-rise building applications, the restoring capacity from tributary gravity loads supported by braced frames can be sufficient to achieve adequate seismic performance, even in regions of high seismicity [9]. The use of such gravity-controlled rocking braced frames (GCRBF) eliminates the need for post-tensioned tendons or base springs, which reduces construction costs. In the case of tendons, the problem of accommodating the high tendon strains resulting from the tendon length being limited to the low building height is also avoided [8]. For seismic retrofit projects, GCRBFs represent a more cost-effective and logical solution as the modifications to the existing structure can be limited to the column bases.

Column uplift and impacts upon rocking of a GCRBF may induce damaging vertical accelerations and member forces in the surrounding building structure. This demand was observed in numerical simulations performed by the authors, as illustrated in Figure 1b for a two-storey building structure located in Vancouver, Canada. For this structure, schematically represented in Figure 1a, beam additional vertical accelerations exceeded the acceleration of gravity, which resulted in increased flexural and shear demands. Column axial loads were increased by a factor of 2.2 compared to that imposed by gravity loads. Vertical accelerations may also cause damage to the building content, which may be critical when building functions must be resumed shortly after a severe earthquake [10]. This additional demand must therefore be considered in the design of new structures or when GCRBFs are used for the seismic rehabilitation of existing structures.

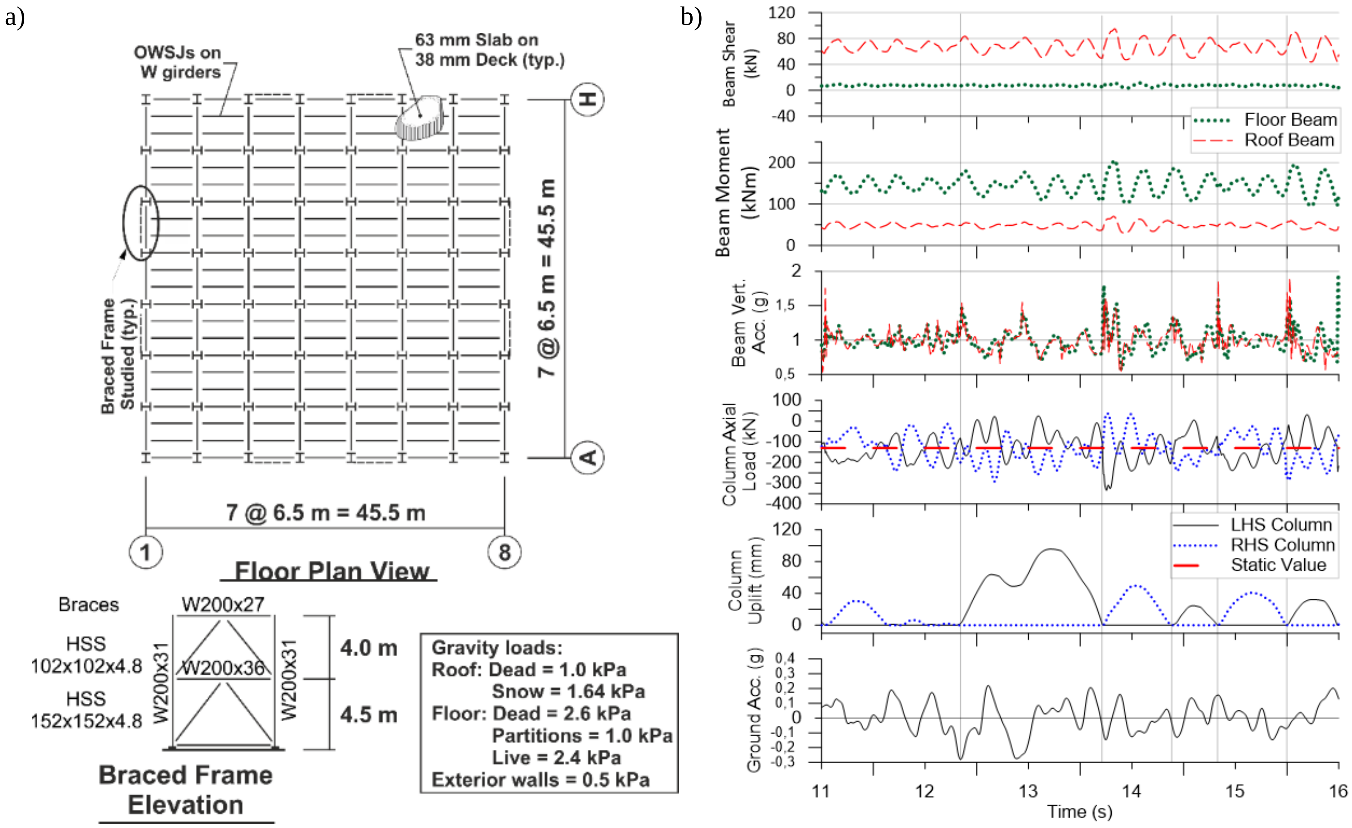

Figure 1—Vertical acceleration and member force demands from the uplift and impact of columns of a GCRBF for a two-storey building in Vancouver, Canada; (a) Structure studied; b) Response history of vertical accelerations and forces under one ground motion [9].

No experimental data is available to confirm the effects of column uplift and impact on the adjacent structure. As part of an ongoing research study on GCRBFs, a test program was performed to examine the possible consequences of column uplift and impact. The tests were performed on a GCRBF column specimen inspired from the building shown in Figure 1. A roof beam and a floor beam supporting representative gravity loads were connected to the test column. During the tests, the column was dropped from gradually increasing initial uplifted positions, and the resulting responses were monitored. This article presents the test program and the results obtained for the accelerations and forces in beams and columns**.**

TEST PROGRAM

Experimental Setup

The test program was conducted in the Structural Engineering Laboratory at Polytechnique Montreal. The test setup is schematically illustrated in Figure 2. A full-scale test model was built from 350W steel to obtain representative values of the accelerations and member forces induced by column uplift and impact in actual GCRBF structures. It included one column of a two-storey rocking braced frame, as well as the beams at the roof and floor levels in the bay next to the rocking frame bay. This configuration was chosen because the peak moment demands would occur in these adjacent members due to the absence of chevron bracing. At their opposite ends, the beams were connected to short W200x27 column segments attached to a fixed base supporting strong column. Design and dimensions were inspired from the two-storey building illustrated in Figure 1. However, reduced storey heights and gravity loads were used due to laboratory physical constraints. Modified member sections were also selected as the design presented in Figure 1 was realized according to past editions of the National Building Code [9]. Beam sections were Class 2 at the roof level and Class 1 at the storey level. Beams were connected to the columns through typical single three-bolt shear tab connections (Figure 3a). The bolts in these connections were manually torqued at the beginning of the tests; no retightening was done during the sequence of uplift tests. Details of the beam-to-column connection are presented in Figure 3a. Gravity loads were applied to the test specimen using concrete blocks suspended under the beams through the use of bolted connections, schematically represented in Figure 2b. As shown, two blocks were used for each beam, each block being simply supported with longitudinally flexible hangers to prevent any composite action between the block and the beams (Figure 2b). The masses were 21.8 kN each on the storey level and 14.9 kN each on the roof level. The GCRBF column was restrained out-of-plane at both levels by means of two steel guides anchored to the strong wall of the laboratory. To minimize frictional effects, a small gap was left between the guides and the column (Figure 2c). The base of the GCRBF column is shown in Figures 2d and 3b. The column base plate did not include anchor rods, horizontal blocking or energy dissipation devices such that unrestrained uplift and impact column responses could be monitored during the tests. The foundation comprised a 305 mm thick reinforced concrete block anchored to the strong floor by means of four pre-tensioned 51 mm diameter high-strength steel bars. A 25 mm thick grout layer was applied between the base plate of the column and the concrete foundation. The measured compressive strengths of the footing concrete and grout were 31.6 and 36 MPa, respectively.

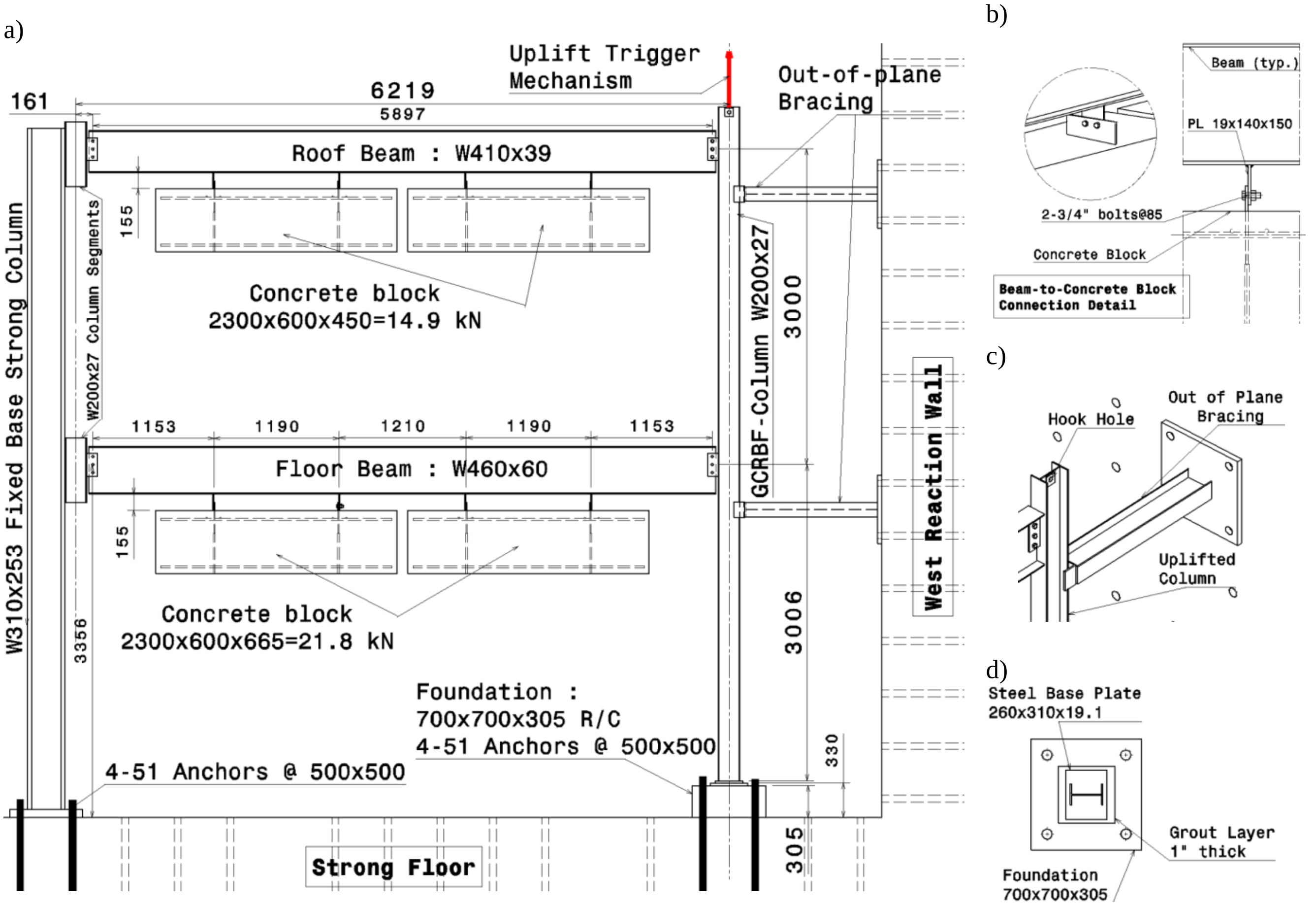

Figure 2—Test setup: (a) Elevation view; (b) Detail of beam-to-mass connections; (c) Detail of the column guides; and (d) Detail of the GCRBF column base plate and foundation.

Uplifting and dropping of the test column was accomplished using a special trigger mechanism mounted on the column top and activated by an overhead crane with two 5-ton hoists servicing the test setup. The trigger mechanism, shown in Figure 3c, was designed to allow a vertical release of the column without any initial velocity or initial rotation. When the mechanism was triggered, the column was free to fall without any friction or any link to the crane.

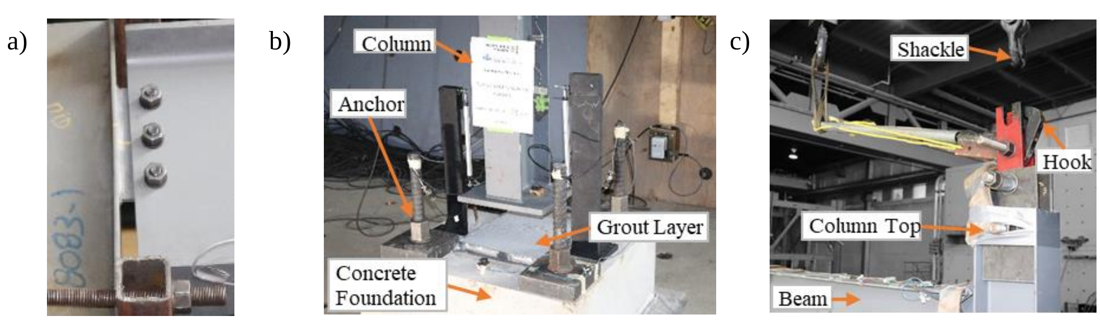

Figure 3—Details of the test specimen: (a) Beam-to-column connection; (b) Base of the uplifted GCRBF column; and (c) Uplift trigger mechanism of the rocking column.

Instrumentation

Figure 4a shows details and locations of the sensors that were used in the test program. Accelerometers were used to measure vertical accelerations at different elevations along the GCRBF column: two 50g accelerometers on the column base plate, ten vertically 10g accelerometers placed at regular spacing along the column flanges and web. In addition, 20g accelerometers were used to measure the vertical accelerations of the foundation block. Three accelerometers were installed at both ends and at the mid-span of each beam to measure vertical accelerations. 3g and 5g accelerometers were used at the roof level and floor levels, respectively. Linear pots were used to measure the vertical displacement of the GCRBF column base. Strain gauges were placed horizontally on beams and vertically on columns to obtain member forces. Axial compression in the GCRBF column was obtained at three elevations from the mean value of the strain gauge readings at these positions. Bending moments were obtained at 5 positions along the beams from curvatures determined with strain gauges placed on the beam flanges. End shears in beams were calculated from gradients in bending moment measured near the beam ends. All data was recorded using an HBM synchronized data acquisition system and processed with Catman DAQ software. Two different acquisition rates were used during the tests: 300 Hz during the dynamic characterization tests and 2400 Hz during the column impact tests.

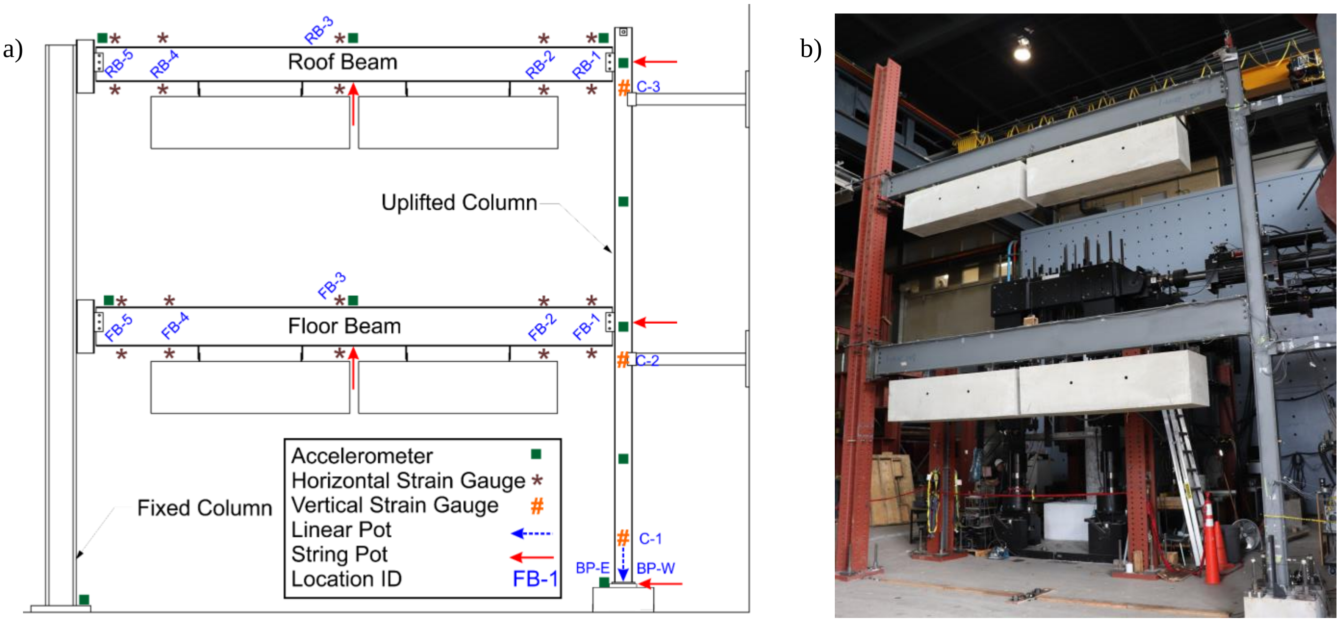

Figure 4— Test setup: a) Instrumentation of the specimen; b) Overall view of the test setup.

Testing Protocol

The test program included a series of six column impact tests performed at gradually increasing initial uplift heights. The test program also included dynamic characterization tests to determine the natural frequencies of the roof and floor beams: one characterization test was performed prior to initiating the column impact test series, and one characterization test was performed after each column impact test. In each characterization test, the GCRBF column was uplifted by 25 mm and dropped to generate vibrations of the beams that were used to determine their frequencies. The six column impact tests were performed by uplifting and dropping the GCRBF column from heights of 25, 50, 75, 100, 125, and 145 mm, respectively. This range was established from previous numerical seismic simulations [9] that showed mean peak uplift values ranging between 30 and 160 mm for two- and three-storey structures located in Vancouver, BC. For safety reasons, it was decided not to go further than 145 mm. The last uplift value induced a 2.2% frame drift angle. In the characterization and column impact tests, lifting of the column was performed slowly and the column was maintained in the uplifted position until the vibrations from the uplift had completely attenuated before being dropped.

Characterization Tests

The first mode frequencies of the beams from the characterization tests are presented in Figure 5a. The frequencies from modal analysis of the specimen are also given for comparison purposes. As shown, the measured first mode frequencies of the two beams correspond well to the theoretical values (residuals of 4.5% and 5% for the roof and floor beams, respectively). For both beams, they remained nearly constant over the test program, although the values slightly decreased as the number of tests performed and the uplift amplitudes increased. This variation can be explained by a progressive reduction of the flexural stiffness of the beam-to-column connections during the test program. As noted earlier, bolt retightening was not carried out.

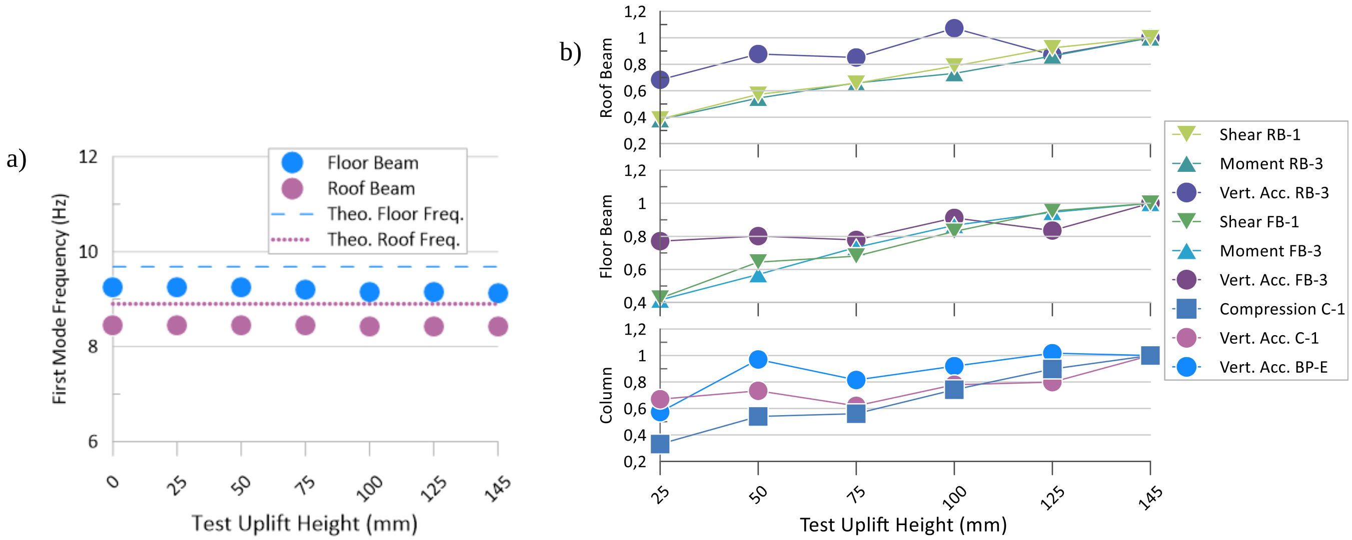

Figure 5—a) First mode beam frequencies from characterization tests; b) Peak response parameters from column impact tests (normalized w.r.t. 145 mm uplift test values)

COLUMN IMPACT TESTS

The response history results from the column impact test performed for an uplift of 125 mm are presented in Figure 6. The locations where response parameters were measured or calculated correspond to the location IDs shown in Figure 4a. Static values are included in all plots; the vertical displacements shown in Figure 6a include displacements from initial uplift. The time values when the first and second column impacts occurred are indicated by the vertical light grey lines. To improve the clarity of the results, all signals have been low-pass filtered with a cut-in frequency of 150 Hz.

The column drop induced negative accelerations at the base plate and in the frame members (Figure 6b to 6d). A strong peak of high-frequency positive vertical accelerations was felt in the base plate and briefly reached a value of 28g. This first impact was followed by a series of 3 rebounds of the column on its foundation. As shown, the height of the rebounds reduced significantly between the first one (45 mm) and the last one (4.5 mm). The triggered peaks of vertical acceleration were of similar magnitude, as can be seen from Figures 6c and 6d. The first two column impacts produced large compression force peaks at the three locations along the GCRBF column (Figure 6e), as well as significant increases of the bending moments and shears in the beams (Figures 6f and 6g). Figure 6h shows the vertical displacement of the storey beam measured at data reading point SB-3. The plots were synchronized so that the impact instant is the same for all tests. As shown, the first oscillation of the beam happened during the same duration, just after the impact. Conversely, the duration of the successive oscillations tended to increase with the height of the drop test: the higher the drop, the longer the oscillations. The amplitude of these oscillations was also proportionally increased with the drop height.

Column Response

In all tests, column impacts induced sharp and large amplitude compression peaks in the columns. Peaks measured at locations C-1, C-2 and C-3 for the test height of 125 mm can be seen in Figure 6e. The presented values are obtained from the mean deformations of 5 strain gauges located at the same elevation along the column. Figure 5b shows the values recorded at C-1 in the other tests. A dynamic amplification factor (DAF), defined as the ratio of the peak axial force demand due to impact to the static force value, can be used to evaluate the effects of column impact on column forces. The static value is the one measured before the tests when the column is at its initial position. It is equal to 48.3 kN. For the test presented, the DAF for the first peak (t = 0.24 s) reached 8.9 at location C-1 near the column base, which is equal to 0.36 times the squash load of the column P_y defined as P_y=AF_y. As shown, the amplitude of the peak is maximum after the first column impact and reduces gradually in the subsequent rebounds. For the case shown, the DAF values are equal to 4.6 for the second impact (t = 0.45 s), and 2.6 for the third one (t = 0.6 s). The magnitude of the DAF reduces as we move up along the height of the column, as can be observed in the magnified view of Figure 6e. For the first impact (t = 0.24 s), the DAF values are 7.2 at the middle level (C-2) and 3.6 at the top level (C-3). It can also be observed that the times at which the first peaks are measured are approximately the same at all three elevations along the column height. Finally, it can be seen from the magnified view that the peak compression load for the first impact in fact comprises two peaks with decreasing amplitudes. The first one (t = 0.25 s) is induced by the first contact between the foundation and the base plate. The second one occurs at t = 0.29 s, while the column is still in contact with the foundation, and is attributed to beam vibration as its coincides with the peaks in shears and moments near beam ends.

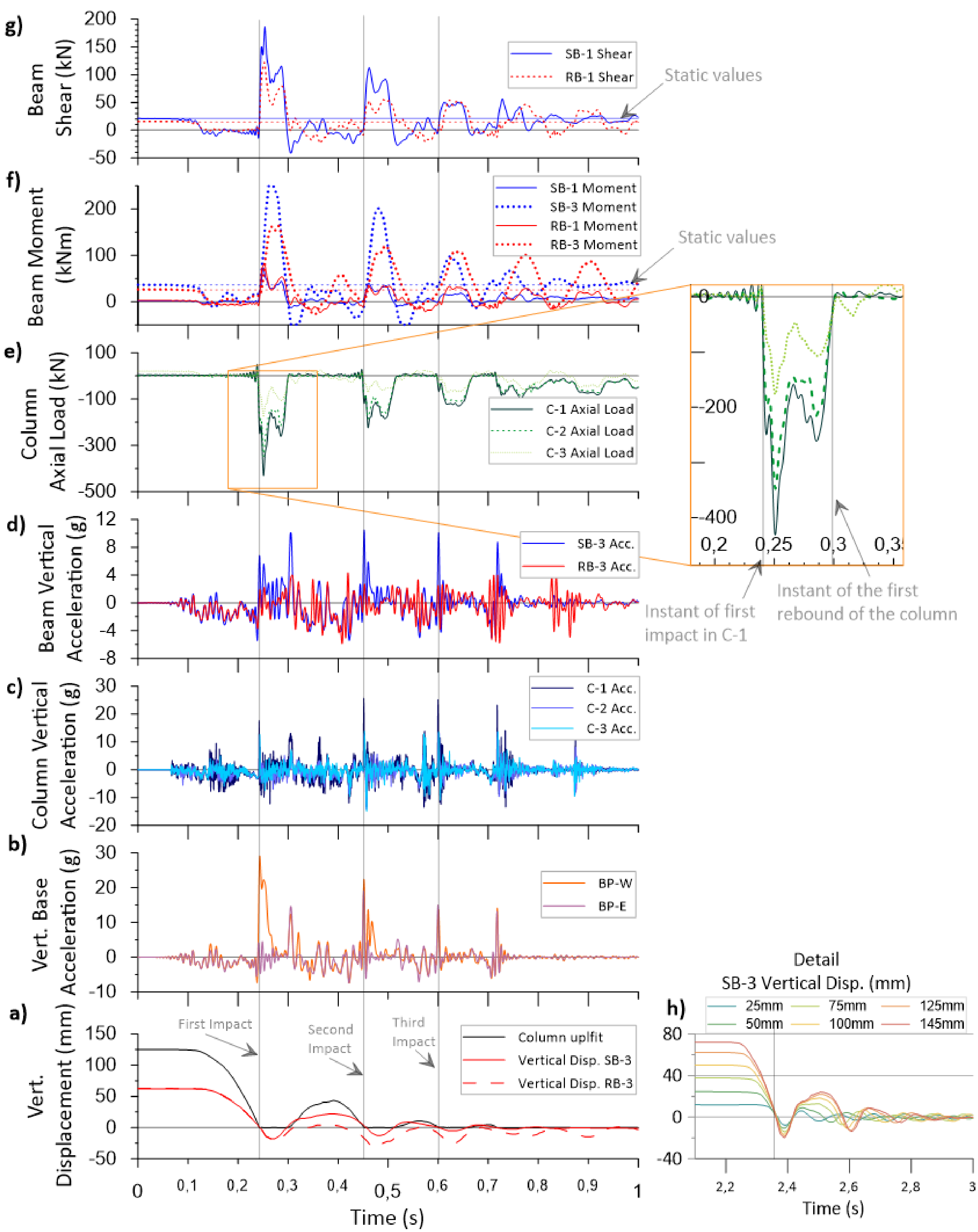

Figure 6—Response history response of the test specimen subjected to 125 mm uplift.

Beam Response

The beam response is illustrated in Figures 6f and 6g. The horizontal lines represent the static value of the bending moment and beam shear. The column drop induced an increase of both the bending moment demand and the shear demand on the beams, due to the inertia forces developed in the beams. The masses carried by the beams tended to keep falling after impact of the column, thus increasing the demands on the beams. As shown in Figure 6f, that increase lasted only as long as the column remained on the foundation. At the storey level at mid-span, the moment demand reached 227 kNm, which is equal to 6.2 times the static value imposed by the masses. In comparison, the plastic moment resistance of the storey beam was equal to 448 kNm. At the roof level, the moment reached 157 kNm, which is also equal to 6.2 times the static value of the moment demand. The plastic moment resistance of the roof beam was 253 kNm. While no permanent deformations of the beam at the storey level were visually observed, the strain gauges located at mid-span on the bottom flange yielded during the last tests. All of the other strain gauges did not show any residual strains.

As illustrated in Figure 6 g, the evolution of the shear force in the two beams followed the same response as described for the bending moment. It suddenly increased when the column impacted the foundation. During that time, the shear was amplified by a factor of 8.4 in comparison with the static shear at both the storey level and the roof level. However, it did not exceed either the nominal shear resistance of the storey beam (841 kN) or of the roof beam (537 kN).

As noted before, both the beam end shears and the beam moments at locations FB-1 and RB-1 are in phase with the compressive load in the column: the peak values of the beam shear are measured at approximately the same time as the peak column compressive loads. The same synchronism is noted between the vertical accelerations measured in the column and the ones measured in the beams.

Overall Response of Frame Members

Table 1 presents the maximum values of the response parameters for the column and the two beams during the six tests that were performed. These parameters are also plotted in Figure 5b. In the figure, the values are normalized to the value obtained during the test with the largest uplift value (145 mm). In the column, the peak axial compression generally increases with the height of the drop; the regression coefficient of the best-linear fit (not plotted for clarity) is R^2=0.98. The evolution of the vertical accelerations is however not as regular, as the regression coefficient are 0.75 and 0.80 for the vertical acceleration in the base plate and in the column, respectively. In the beams, vertical accelerations also generally increase along with the height of the uplift, though the increase rate foes vary between tests. However, shears and bending moments in beams increase almost linearly with the height of the drop. For instance, the regression coefficients of the best-linear fits are R^2=0.98 and 0.99 for the moments at the floor and roof levels, respectively.

Table 1 Peak response parameters from column impact tests

Test Height (mm)	Max. Vert. Base Acc. (g)	Max. Col. Vertical Acc. (g)	Max. Storey Beam Acc. (g)	Max. Roof Beam Acc. (g)	Max. Column Comp (kN)	Max. Sto. Beam Moment (kNm)	Max. Roof Beam Moment (kNm)	Max. Sto. Beam Shear (kN)	Max. Roof Beam Shear (kN)
25	16.4	10.7	9.6	3.4	159	99.8	69.6	82.7	50.6
50	27.7	11.7	9.98	4.37	259	136	99.1	125	75
75	23.3	9.85	9.69	4.24	269	176	120	132	86
100	26.3	12.4	11.35	5.34	356	208	133	161	103
125	29.1	12.7	10.4	4.33	431	227	157	185	121
*145**	*28.6*	*15.9*	*12.5*	*4.98*	*480*	*240*	*182*	*194*	*131*

** Test value used as a reference for the normalization shown in Figure 5b.*

Demands on the foundation

The second objective of the test program was to examine the behaviour of the foundation during the impact of the column. Both the grout layer and the foundation were closely inspected between every test to see whether they had been damaged by the previous test. The results showed that the foundation did not experience any noticeable damage during the tests: no cracks were observed. The grout layer was found to have one crack at the end of the last test. This suggests that foundations should be inspected after a major earthquake and possibly repaired, given the fact that the strength of the tested foundation was representative of what could be found in a common building. It is important to once again note that no ED device was added to the base of the column for this test program; it is anticipated that a device of this type could help reduce the accelerations felt by the foundation and consequently the potential for damage to the concrete and grout.

Demands on the beam-to-column connections

The last objective of the test program was to examine the effects of the impacts on the beam-to-column connections. No major visual damage was observed in the connections after the tests had been completed. Some local bearing was seen in the bolt holes in the column shear tabs and in the beams. The threads of the bolts were slightly crushed and left marks on the steel of the beams. Altogether, these observations tend to show that the connections were not significantly damaged by the tests. However, further investigation should be conducted on the behaviour of bolted connections subjected to axial impact loading, in order to confirm this observation.

CONCLUSION

An impact test program was successfully developed to examine the consequences of column impact occurring in rocking braced frames subjected to seismic ground motions. The tests aimed at quantifying the force demands in the frame members due to the impact with focus on the compression loads induced in columns and flexure and shear demands in beams. Another objective was to observe whether the impact loading could have detrimental effects on the structural integrity of beam-to-column connections and foundations.

A full-scale test setup was designed to represent a portion of the frame adjacent to the rocking braced bay of a two-storey building. Column uplift gradually increasing from 25 to 145 mm were imposed, based on the demand expected for Vancouver, British Columbia. Tests were also conducted to characterize the dynamic properties of the specimen to assess the cumulated damage experienced in the sequence of tests. The results showed that the impact caused by rocking can induce significant peaks of the compressive load and localized yielding in the rocking column, but that this demand would happen so quickly that the structural integrity of the column would not be affected. The frequency content of these compression loads was indeed at least three times higher than the frequency of the column itself. A nearly linear correlation was observed between the drop height and the peak values of column compression as well as peak shears and moments in beams at both levels. The beam-to-column connections and foundation experienced only minor damage including slight bolt hole elongation and one crack that developed in the grout during the test with largest uplift height, indicating that short duration impacts were not sufficient to cause substantial damage in the structure. Further tests should be performed on specimens with realistic floor slabs to validate the consequences of column impact on the structural integrity of a GCRBF during major earthquakes.

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