NUMERICAL MODELLING AND FULL-SCALE EXPERIMENTAL VALIDATION OF THE GAPPED-INCLINED BRACING SYSTEM

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ABSTRACT

The collapse of soft-storey structures during seismic events has been observed from many post-earthquake reconnaissance missions. Soft-storey buildings are often inadvertently created to meet architectural demands, such as dwelling units located above a parking area at the ground level. During a seismic event, damage is concentrated to the weaker more flexible ground-floor columns, leading to collapse of the building. The gapped-inclined braces (GIB), a soft-storey retrofit proposed by Agha Beigi et. al (2014), was developed to address the above issues. These braces are installed only at the ground floor. The unique gap element and close-to-the-column installation allow the soft-storey to form while preventing collapse of the building; this system takes advantage of the soft-storey, creating an isolation layer at the ground-floor, protecting the upper floors of the building from seismic forces. A three-phase, full-scale experimental programme was recently completed at the University of Toronto. Phase I focused on the response of a single GIB. The second and third phases compared the response of a single bay, single storey reinforced concrete (RC) frame representative of the ground floor in a soft-storey building: a conventional frame and a GIB-retrofitted frame. The results showed a significant improvement in the displacement capacity of the retrofitted frame. The RC frames were modelled using VecTor2, a nonlinear finite element analysis program for RC elements under extreme loading conditions. This paper compares some of the experimental and numerical results.

Keywords: Soft-Storey; retrofit; full-scale experimental; numerical modelling; VecTor2

1. INTRODUCTION

Post-earthquake reconnaissance has continually highlighted the poor seismic performance of ground-level soft-storey buildings (Mahin et al. 1979; Zhao et al. 2009). A preliminary report of the Mexico City earthquake in 2017 indicated that 60% of the collapsed buildings had a soft-storey configuration (Galvis et. al, 2017). The insufficient stiffness and strength at the ground-floor, which is often the result of meeting architectural demands, such as a parking area at the ground-floor, concentrates damage to the ground-floor columns. The gravity load from the floors above induce second-order forces through the displacement of the first-floor columns, leading to the collapse of the building. Galvis et al. (2017) noted that much of the infrastructure built after the 1985 Mexico City earthquake performed well, suggesting that newer construction adhere to modern seismic design philosophies. On the other hand, it shows a lack in the assessment or intervention of the existing building stock. There is an evident need for affordable retrofitting strategies to prevent similar future disasters, which holds true for many seismic prone regions of the world.

Agha Beigi et. al (2014) proposed the gapped-inclined bracing (GIB) system to address the issues above. The GIBs are installed adjacent to the first-floor columns in a near vertical configuration with a specially designed gap element. The gap element ensures that the original stiffness of the building is unchanged and allows the GIB to transfer only compressive forces. The GIB is activated through the lateral

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displacement of the ground floor. After activation, the GIBs relieve the axial load on the existing columns as the building sways laterally, increasing the displacement capacity of the building, while preventing collapse. The GIBs are installed only at the ground floor and do not hinder the architecture function of the building; the retrofit promotes the isolation benefits provided by the soft-storey, while preventing collapse. Agha Beigi et al. (2016) performed a cost-benefit analysis of the GIB retrofit compared to more conventional techniques, such as cross-bracing, and found that the GIBs far outweighed the traditional methods.

Furthermore, a three-phase, full-scale experimental programme of the GIB system was recently completed at the University of Toronto. The first phase tested a single brace. Phases II and III compared the reverse cyclic response of two single bay, single storey RC frames representative of the ground-floor in a soft-storey structure: a conventional frame and a GIB-retrofitted frame. The results showed a significant improvement in the displacement capacity of the retrofitted frame. The RC frames were modelled using VecTor2, a nonlinear finite element analysis program for RC elements. This paper compares selected experimental and analytical results.

2. PHASE I: GIB-ASSEMBLY

The GIB assembly is composed of three components: a lower connection, a threaded adjustment screw, and an HSS top connection. The threaded adjustment screw threads onto the lower connection and acts as a guide for the HSS top connection, such that compressive forces are transferred between the elements. Figure 1 shows a schematic of the GIB assembly. Three gap sizes were tested, and the results were as expected: (1) increased gap sizes lead to larger inter-story drifts before GIB activation, (2) the GIB acted as a compression-only element, (3) the GIB responded elastically when loaded to 600 kN, and (4) for a given GIB angle, increasing the gap size results in a lower lateral resistance. Further details of the experimental results can be found in Salmon et. al (2018).

3. PHASES II AND III: TEST SPECIMEN AND LOADING PROTOCOL

Phases II and III compared the hysteretic response of two RC frames, representative of the ground-floor in a soft-storey building. The first frame, denoted as conventional, was used as a baseline response. The second frame was built identical to the conventional frame but included the GIB system, which was installed after casting. Each frame was subjected to a quasi-static lateral loading protocol and constant axial load of 500 kN applied to each column. Figure 2 shows the loading protocol for each frame, and the ‘X’ in the figure denotes failure of the conventional frame. Figure 3 shows the reinforcement details of the RC frame test specimen. Further detail on the specimen can be found in Salmon et al. (2018).
The RC frame was positioned in a vertical testing frame at the University of Toronto. Figure 4 shows the test setup. A constant axial load of 500 kN was applied through a slide bearing axial load application system (ALAS). The ALAS was positioned between the RC column stubs and a reaction beam. The reaction beam was supported by two exterior reaction columns, which were pin-connected to the strong floor, and four interior reaction columns, which were positioned adjacent to the ALAS and anchored into the concrete foundation. Two 1000-kN actuators were attached to the RC frame at mid-span of the beam and reacted against a strong wall to apply lateral load. In-plane braces resisted the in-plane translation of the test setup. Various out-of-plane pieces were used to control the movement of the test specimen and test setup.
4. PHASE II EXPERIMENTAL AND NUMERICAL RESULTS

4.1 Experimental Results

The hysteretic response and a photo of the failed specimen are shown in Figure 5 (a) and (b), respectively. The black dot in Figure 5 (a) denotes failure of the frame. Strength degradation was observed beyond ±1.5% until reaching failure after the first cycle at 4% drift. The lateral resistance was governed by plastic hinging at the top and bottom of the RC columns, indicative of a soft-storey failure. Little-to-no damage was observed in the beam-column joints, and no yielding was observed in the beam. Further detail on the experimental results can be found in Salmon et al. (2018).
4.2 VecTor2 Numerical Model

The conventional frame was modelled using VecTor2, a nonlinear finite element analysis program that uses membrane elements to analyze two-dimensional concrete structures. The program was formulated based on the constitutive relationships of the disturbed stress field model (Vecchio F. J., 2000), a refinement of the Modified Compression Field Theory (Vecchio & Collins, 1986). The program uses a smeared rotating crack, iterative secant stiffness procedure to achieve convergence at each load step. The software has been developed through more than 30 years of research at the University of Toronto (Vecchio 1989) and has shown to accurately model RC structures under a variety of loading scenarios, including its early formulations for monotonic loading to later revisions that include cyclic and temperature loading (Palermo & Vecchio, 2007). Figure 6 shows the VecTor2 model of the conventional RC frame.

The concrete was modelled using isoparametric quadrilateral elements, which were needed to consider geometrical nonlinear effects. The longitudinal rebar reinforcement was modelled as discrete truss bar elements; the transverse and out-of-plane reinforcement were smeared within the concrete elements. Details of these elements are provided in (Wong et. al, 2002). The analysis parameters were set to the Advanced option with VecTor2, which tends to align with lightly reinforced concrete elements.

Six concrete types, four reinforcement materials, and two structural steel loading plate were used to
model the conventional frame. The difference between the concrete material types was the amount of smeared reinforcement, which can be calculated from the reinforcement details given in Figure 3. Structural steel loading plates were used at locations of concentrated loads or displacement, i.e. at the top of the column stubs and at mid-span of the beam. The yield strength of the steel was specified as 300 MPa.

Rebar buckling was modelled using the equations outlined in Wong et. al (2002), and was applied to all longitudinal reinforcement, except the 600-mm lap splice region at the bottom of the columns. Supplemental confining reinforcement were used at the top and bottom of the columns, representing the confinement provided by the foundation and beam, respectively.

The VecTor2 model described above was subjected to one cycle at each displacement amplitude of the conventional frame. The model was subjected to only one cycle because it was found that the degradation of the concrete was too large for the repeated cycles; this issue will be addressed in future work. Figure 7 compares the numerical and experimental results. The initial stiffness and peak strength were well captured by the model. The stiffness degradation beyond ±1.5% drift was also accurately modelled. Failure was governed by crushing of the concrete at the base of the column, after a half cycle at 3% drift. The unloading stiffness differed between the numerical and experimental results. The analysis showed a significant amount of cracking at the top of the beam, where the reinforcement changed from six bars to two. Crack widths were greater than 8 mm were determined analytically, which was not observed experimentally. The lateral resistance of the frame was largely governed by plastic hinging at the top and bottom of the columns, as observed experimentally. In general, cyclic degradation of the concrete was greater in the model than the test; however, it was felt that the model captured the response of the frame well.

Figure 7. Comparing the conventional RC frame to VecTor2 results

The crack pattern predicted by the numerical model and the cracking at the top of the column and beam-column (BC) joint at 3% drift are shown in Figure 8. The VecTor2 model showed a significant amount of cracking in the BC joint, which was not observed experimentally.
Figure 8. Comparing damage states at 3% drift - (a) VecTor2 crack pattern; (b) top of the north column; and (c) south BC joint

5. PHASE III EXPERIMENTAL AND NUMERICAL RESULTS

5.1 Experimental Results

An identical RC frame was built for Phase III. The GIBs were subsequently installed in the frame. The GIB parameter, such as the GIB angle and gap size, were determined using VecTor2 following the procedure outlined in Agha Beigi et. al (2014). The chosen parameters are summarized in Table 1.

Table 1. Summary of GIB properties for phase III testing.

<table>
<thead>
<tr>
<th></th>
<th>Interior GIB</th>
<th>Exterior GIB</th>
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<tbody>
<tr>
<td>$F_y$</td>
<td>64 kN</td>
<td>73 kN</td>
</tr>
<tr>
<td>$F_u$</td>
<td>41 kN</td>
<td>45 kN</td>
</tr>
<tr>
<td>$P_{\text{initial}}$</td>
<td>434 kN</td>
<td>564 kN</td>
</tr>
<tr>
<td>$P_u$</td>
<td>113 kN</td>
<td>113 kN</td>
</tr>
<tr>
<td>$\theta_{\text{ult}}$</td>
<td>4%</td>
<td>4%</td>
</tr>
<tr>
<td>$\Delta_{\text{GIB}}$</td>
<td>320 mm</td>
<td>290 mm</td>
</tr>
<tr>
<td>$\Delta_{\text{Gap, design}}$</td>
<td>3.1 mm</td>
<td>3.0 mm</td>
</tr>
</tbody>
</table>

Figure 9 (a) and (b) show the experimental results of the hysteretic response and vertical reaction of the retrofitted frame, respectively. The combined response (light grey) is decomposed into the contributions of the columns (blue) and the GIBs (red). Although degradation was observed in the columns, the combined response showed only slight degradation with repeated cycles at 4% drift. The system provided a stable hysteretic response to 4% drift. The GIBs activated near $\pm 1\%$ drift and carried nearly the fully applied axial load by the end of the 4% drift cycle. Further details on the
experimental results can be found in Salmon et al. (2018).

Figure 9. Experimental response of the retrofitted frame - (a) Hysteretic and (b) Vertical reaction

5.2 VecTor2 Numerical Model

A VecTor2 model was created of the retrofitted specimen; the model is the same as the conventional frame except a steel bearing plate and compression-only truss elements were added to represent the GIB-to-frame connection and GIB, respectively. The model was subjected to one cycle at each drift amplitude; the experimental and numerical results, up to 3% drift, are compared in Figure 10. The initial stiffness, unloading stiffness, and peak strength of the frame were well captured analytically. With heavily degraded columns after completing the 4% drift cycle and sustaining complete crushing at the top of the columns, the program had convergence issues and the analysis stopped.

Figure 10. Retrofitted Frame Hysteretic Response

The full hysteretic response (light grey) determined numerically is decomposed into the contribution of
the columns (blue) and the GIBs (red), as shown in Figure 11. Comparing Figure 9 and Figure 11, it is shown that the model overestimates the column strength degradation, which was also shown in the analysis of the conventional frame. However, similar trends are obvious between the experimental and numerical analyses. The lateral resistance of the GIBs reaches a maximum at roughly 4% drift; As the degradation of the columns increased, the GIBs activate at earlier drifts in subsequent cycles. The damage sequence was well captured by the model, with the exception of faster degradation in of the column capacity.

![Hysteretic Response - VecTor2](a)![Vertical Reaction - VecTor2](b)

Figure 11. VecTor2 results of the retrofitted frame – (a) Hysteretic and (b) Vertical reaction

Figure 12 (a) and (b) compare the numerical and experimental damage state, respectively, of the retrofitted frame at 4% drift. The VecTor2 model showed a significant amount of cracking at the top and the bottom of the RC columns, which was also observed experimentally (Figure 9 (a) and (c)). The model indicated yielding and a significant amount of cracking at the top of the RC beam at the location where the reinforcement changes from six rebar to two, which did not occur during testing until 5% drift. With the GIB carrying the fully applied axial load, the offset between the GIB and applied load induces a moment in the beam sufficient to cause yielding.

![Damage sequence](a) ![Experimental setup](b)
Figure 12. Damage state of the retrofitted frame at 4% drift - (a) VecTor2; (b) whole frame; and (c) top of the north column

6. PHASE II AND III EXPERIMENTAL COMPARISON

The retrofitted specimen was subsequently cycled up to 7% drift, which was the limits of the test setup. The hysteretic response and vertical reaction of the GIBs are shown in Figure 13 (a) and (b), respectively. After repeated cycles, the vertically applied load was completely carried by the GIBs; however, collapse was prevented. Beyond 5% drift, with the GIBs carrying the fully applied axial load, a significant amount of cracking was observed at the top of the beam, where the rebar changed from six bars to two. After repeated cycles, the concrete at the top of the south column had completely degraded, and a roughly 300-mm gap was visible between the beam and the top of the column. Figure 14 shows a photo of the damaged RC frame after testing.

Figure 13. Experimental results of the retrofitted frame – (a) Hysteretic and (b) GIB vertical reaction
Figure 14. Photo of the failed test specimen

Figure 15 compares the experimental hysteretic response of the conventional and retrofitted RC frame. As shown the initial stiffness and peak strength match well, with a slight increase in strength in the retrofitted frame. The retrofitted frame displayed a much more stable response up to 4% drift, and collapse of the frame was completely prevented.

Figure 15. Comparing the conventional and retrofitted frames

7. CONCLUSION

A three-phase, full-scale experimental programme of the GIB system was completed. Phase I focused on the structural response of a single GIB; the results were as expected and confirmed four aspects of the system: (1) the GIB responds as a compression-only element; (2) larger gaps correspond to increased drifts before GIB activation; (3) the GIB responds elastically when loaded to 600 kN; and (4) for a given GIB angle, increasing the gap size reduces the lateral resistance of the brace.

Phases II and III compared the response of two full-scale RC frames representative of the ground-floor in a soft-storey building: a conventional frame and a GIB-retrofitted frame. The displacement capacity of the RC frame was significantly improved with the GIB system. The conventional frame showed strength degradation beyond ±1.5%, but the retrofitted frame showed a stable response to ±4% drift. The retrofitted frame was subsequently cycled up to 7% drift, marking the limits of the test setup. After
repeated cycles at 7% drift, collapse was not observed. The RC frames were modelled using VecTor2. The models were able to capture the initial stiffness and peak strength of both frames well. The unloading stiffness of the retrofitted frame was well captured. Generally, the cyclic degradation of the concrete was greater in the numerical model than observed experimentally. Detailed numerical work is ongoing and will be published in future papers.

8. REFERENCES


