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Collapse Simulation of CFRP Retrofitted Column using Hybrid Testing Technique

Yassamin K. Faiud Al-Ogaidi*, John Wilson**, Javad Hashemi ***, Riadh Al-Mahaidi***, and Kamiran Abdouka***

* Dept. of Civil Engineering, College of Engineering, University of Duhok, Duhok, 42001 Kurdistan Region, Iraq.

** Faculty of Science, Engineering and Technology, Swinburne University of Technology, Melbourne 3122, Australia.

*** Faculty of Science, Engineering and Technology, Dept. of Civil and Construction Engineering, Swinburne University of Technology, Melbourne 3122, Australia.

*Email: yassamink.faiud@uod.ac; Corresponding author.

ABSTRACT

The performance of Reinforced Concrete (RC) structures and their collapse safety experiencing moderate to high-magnitude seismic excitations might not be fully understood. Factors related to the high levels of uncertainties in ground motion content, structural modelling and design uncertainties, or to the empirical nature of design codes and standards affect the collapse assessment of RC structures. Therefore, experimental test researches are very important in verifying and improving the accuracy of structural performance predictions and highlighting the actual behavior during seismic excitation. In this paper, the test results of a full-length damaged RC column retrofitted with Carbon Fiber Reinforced Polymer (CFRP) sheets and then retested using pseudo-dynamic experimental testing approach through the state-of-the-art hybrid simulation testing facility, referred to Multi-Axis Substructure Testing (MAST) system at Swinburne University of Technology are presented. A comparative collapse assessment of the initial and retrofitted column showed more ductile column response

reaching higher drift ratios. In addition, the experimental test results displayed clearly the effectiveness of CFRP sheets under increasing intensity of earthquake demands in restoring the collapse resistance of the retrofitted RC column by altering concrete failure type.

Keywords: CFRP sheets; experimental methods; hybrid simulation; multi-directional loading; RC column.

INTRODUCTION

The seismic collapse assessment of structures in numerous post-earthquake investigations in addition to many experimental researches have mainly attributed the structural collapse of RC structures to losing the lateral loading capacity of their vertical load carrying members (Otani, 1999). Corner columns compared to other vertical elements in RC structures are subjected to the smallest amount of gravity loading during earthquake excitation while enduring the largest magnitude of axial loading. Normally, the eccentric loading of RC corner columns will result in biaxial bending that affect their failure mechanism. Furthermore, the biaxial lateral loading of earthquake motion largely affects the seismic response of multistory frame buildings with greater impact on columns. RC columns failure and structural damage have been reported to include inadequate shear detailing and confinement, insufficient bond and anchorage slippage as well as to inadequate bidirectional capacity (Varum, 2003 & Rodrigues et al., 2013).

To validate and improve the accuracy of assessing the structural response and analytical prediction of seismic performance, experimental laboratory testing of structures and structural elements can be used; three types of seismic experimental testing are available. The 1st type includes applying a simplified quasi-static cyclic loading pattern through imposing seismic demands at slow rate; this test may not present the actual behavior during earthquake ground motion (Park, 1989, Schellenberg et al., 2006 & Charlet, 2007). The performance of complete structural system may be verified using shaking table test technique which use real ground motion records. However, using scaled structure prototypes restricted by shaking tables capacity in addition to the influence of scaling material between concrete and reinforcement bars affect significantly on estimating the actual seismic response. The limited experimental data on the response of RC columns subjected to complex time-varying 6 Degree Of Freedom (6-DOF) has showed that using Hybrid Simulation (HS) technique; this test is evolved from the pseudo-dynamic testing method, is considered as an alternative cost-effective testing technique that viably assess the dynamic performance of structural systems. Large structures are analytically modelled in the computational substructure model while the physical substructure performance in the HS test is experimentally investigated under real-time loading; domain decomposition (Shao et al, 2010 & Hashemi et al, 2017).

Consequently, to address the influence of continuously varying three dimensional actions of earthquake ground motions and for the purpose of testing Three-Dimensional (3D) large scale structural component, the MAST system at Swinburne University of Technology, Melbourne, Australia was utilized to investigate the seismic response of two full length limited ductile RC corner columns subjected to actual ground motion records. The first story RC corner column specimen (RC#1) was tested under the effect of the Imperial Valley ground motion acceleration records in the two orthogonal directions. Then the damaged RC#1 column was retrofitted with CFRP sheets and retested as RC column specimen (RC#2) to examine the retrofitted column's axial and lateral strength capacity under biaxial earthquake demands of the same ground motion records.

NUMERICAL FRAMEWORK

To conduct the HS test, the hybrid numerical model included a half scale case study multistory Ordinary Moment Resistant Office Frame building (OMRF). The limited ductility symmetrical five story frame has a scaled 1st story height; h_l= 2.5m and a typical story height; h_{typ} = 2m. These type of construction arrangements are common in areas of low-moderate seismicity zones like Australia in which are developed and detailed in accordance with the Australian building design codes (AS3600, 2009 & AS1170.4 2007). Figure 1 shows the frame building's reinforced beams and columns configurations. The OMRF elements are modeled using *BeamWithHinges* element type that considers the axial and bending loading using OpenSees software (OpenSees, 2015). The nonlinear behavior followed the distributed plasticity concept in which the plastic behavior is assumed to occur at finite length at both beam-column element ends.

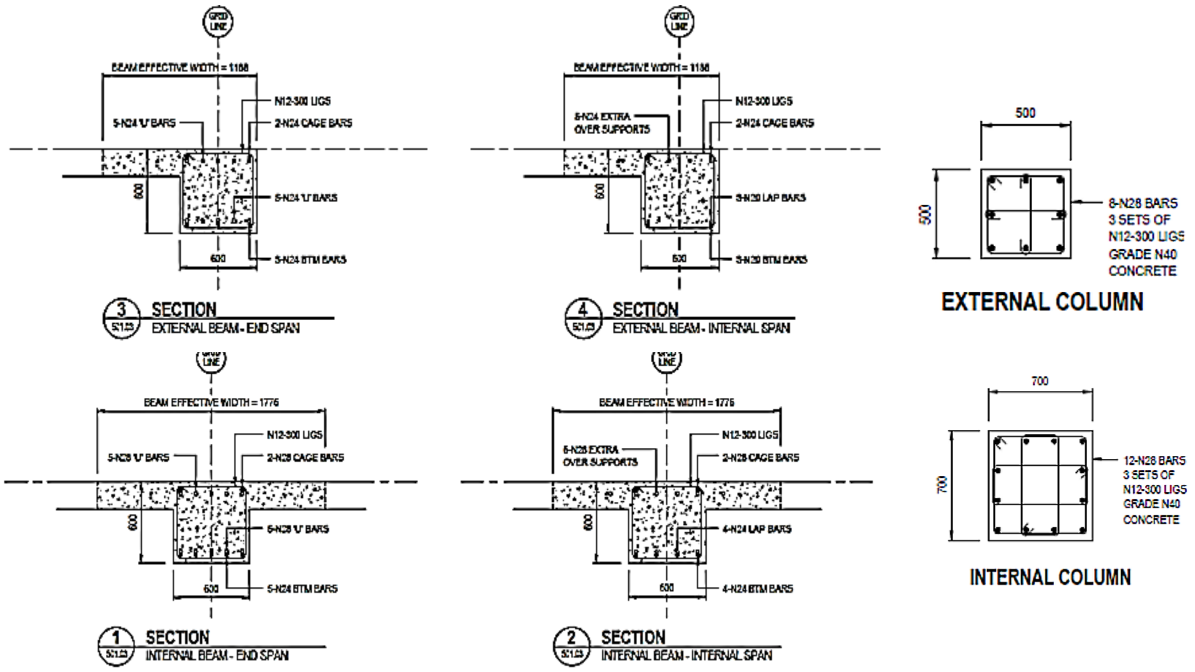


Figure 1 Beams and column reinforcement details of the OMRF building

The lumped plasticity analytical model followed the peak-ordinated material hysteresis response based on the modified Ibarra-Medina-Krawinkler (IMK) deterioration model (Ibarra & Krawinkler, 2005) for the flexural beam-column behavior. This type of modeling used the backbone curve model which is modified by Ibarra et al (Ibarra et al., 2005) to consider both strength capping and residual strength conditions as can be seen in Figure 2.

The HS test components included the numerical substructure model of the frame building fully modeled in OpenSees software; the computational model considered forces such as inertial and damping forces, gravity loading and dynamic loading in addition to the second-order effects while the physical part is presented by the RC corner column as shown Figure 3.

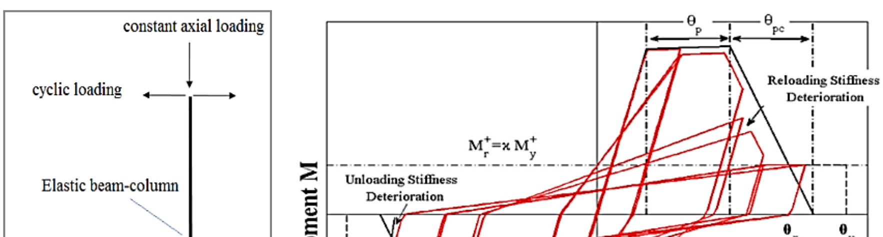


Figure 2 Distributed plasticity model with peak-oriented hysteresis response following finite length concept (OpenSees, 2015)

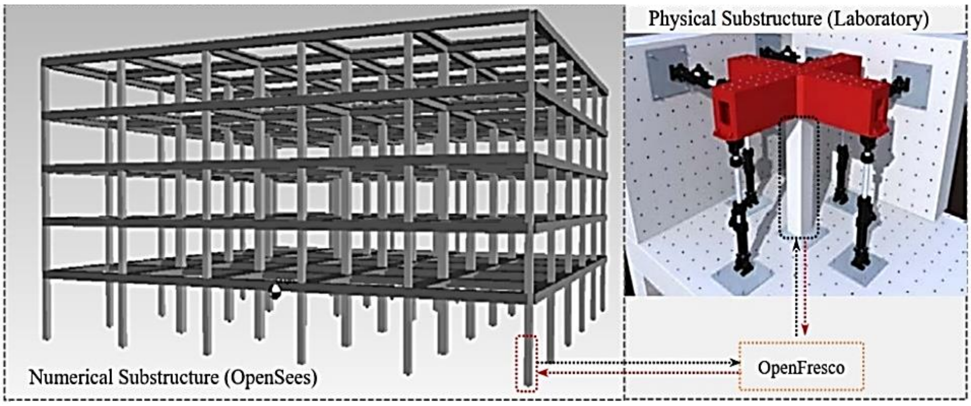


Figure 3 HS test components including numerical and physical substructures (Hashemi et al, 2014)

EXPERIMENTAL TEST SETUP

The column specimen was cast in a heavily reinforced upper and lower blocks; a full description of the RC column specimen reinforcement detailing provided in Figure 4. The test was carried out at the Smart Structures Lab (SSL), Swinburne using the MAST system illustrated Figure 4 as well. in which consisted of an L-shaped strong-wall, a strong-floor reaction system highly stiff steel cruciform and eight high-capacity hydraulic actuators with a capacity of 1MN for the vertical actuator and 0.5MN for the horizontal ones. The longitudinal reinforcement of the test specimen consisted of four 16mm deformed steel bars with a specified yield strength of 500 MPa. While the column specimens' transverse reinforcement included 6mm plain bars distributed equally at 175mm spacing throughout the total height of the column. Two batches of ordinary normal weight concrete were used for casting with an average concrete compressive strength of 37.7MPa at 28 days; it's worth to mention that no scaling was conducted for the material to overcome any differences in the stress field. The OpenFresco (2015) framework was utilized to perform the HS test through enabling a common framework with OpenSees (2015) to support conducting such kind of tests.

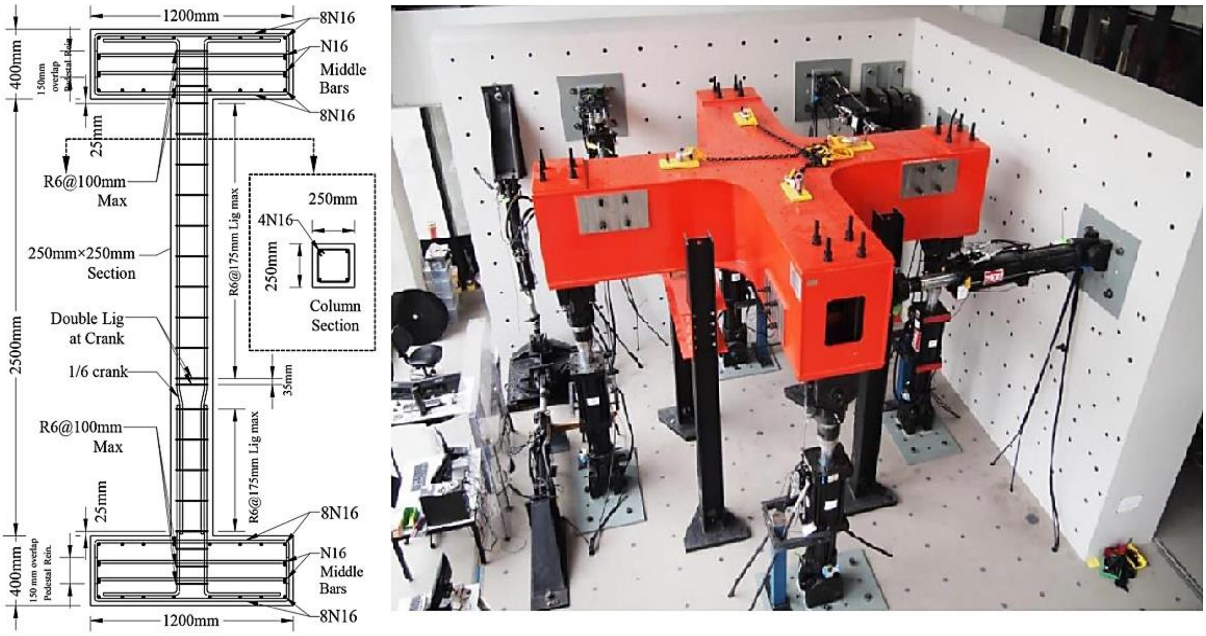


Figure 4 Experimental test setup and design details of the full length RC column

EARTHQUAKE RECORDS AND EXPERIMENTAL TESTING PROGRAM

The experimental testing program started by subjecting the numerical frame model; computational substructure, to a sequential earthquake of four different levels of intensities with scale factors of 0.5(g), 3(g), 5(g), and 6(g) of the 1979 Imperial Valley at El Centro station acceleration records in the two orthogonal directions. The scales were selected to capture the whole seismic performance of the frame model from the elastic range level up to collapse prevention level. The selection of both ground motion records and levels of intensity were based on the analytical simulation of the OMRF model with selected seismic records inputs suites to match the model design spectrum and the performance of the model while performing Incremental Dynamic Analysis (IDA), respectively.

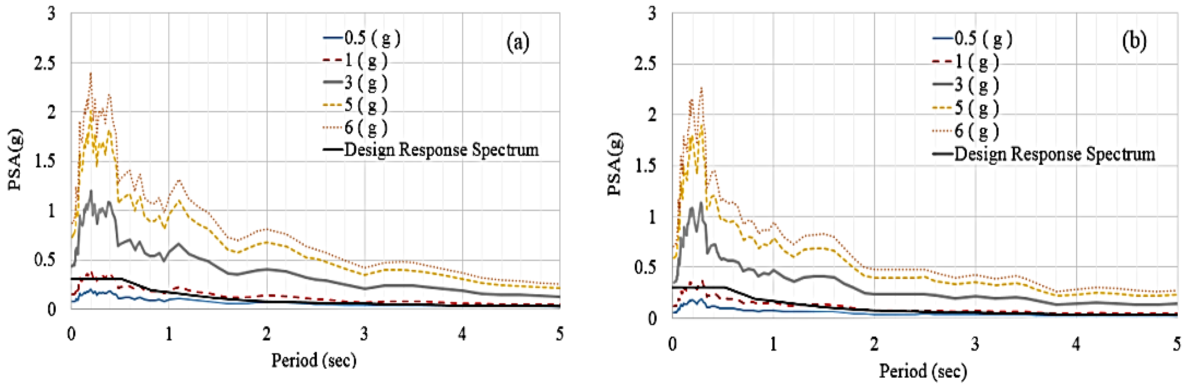


Figure 5 Scaled spectral acceleration of the Imperial Valley 1979 record for both (a) major and (b) minor directions of loading

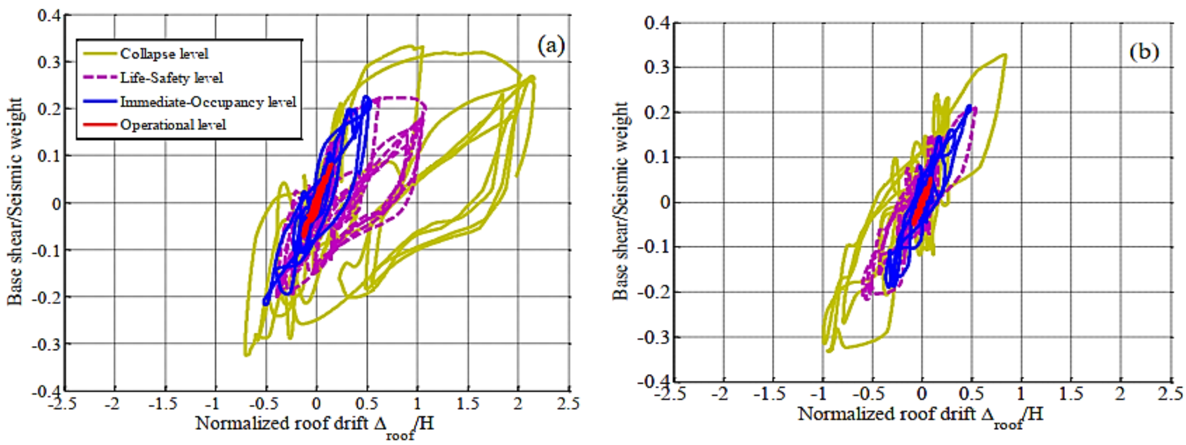


Figure 6 Numerical OMRF model response while conducting IDA in the (a) major and (b) minor directions of loading

The relationship between the Pseudo-Spectral Acceleration (PSA) to the spectral period is illustrated in Figure 5 which shows the acceleration response spectra of the selected scales. Figure 6 illustrates the frame model analytical performance during the four performance levels suggested by the ASCE/SEI41-6 (2007); operational, immediate occupancy, life safety and collapse prevention levels with 5% damping.

RC # 1 COLUMN EXPERIMENTAL TEST RESULTS AND RETROFITTING FRAMEWORK

The RC#1 corner column was tested to structural frame collapse failure under different scale intensity levels of biaxial components of the Imperial Valley acceleration records combined with the influence of variable axial loading that ranged between 22.6% in compression and 6.6% in tension obtained through the MAST system (It worth to mention that RC#1 was originally subjected to a constant compressive axial loading ratio of about 8.5%) (Fig. 7a). The experimental test results noticed in Fig. 7(b) clearly showed the influence of biaxial demands on the lateral force-drift relationship of RC#1 hysteresis performance with high level of strength degradation in the major direction compared to the minor direction of loading. Thus, more damage was observed in the major direction when reaching maximum drift ratio of 6.42% while the minor direction experienced 2.2% drift ratio.

(a) Axial force-bending moment diagram for RC#1 column

(b) Hysteretic responses of RC# 1 column specimen

Figure 7 Experimental axial loading ratio variation and its influence on the hysteresis response of RC#1 column

Longitudinal reinforcement bars yielding and buckling was particularly observed during the test in the lower part at corners when reaching a frame model drift ratio of 2% which was taking place when applying scale 5 (g) of the Imperial Valley records.

The steel bars damage of RC#1 was concentrated at 150mm from the column specimen's both ends associated with concrete cover spalling and core crushing as being demonstrated in Figure 8 especially in the major direction of loading. However, the minor direction experienced the spread of flexural crack at horizontal reinforcement locations with lower level of damaged concrete. Furthermore, there was no evidence of longitudinal and horizontal reinforcements bar fracture or opened stirrups upon the end of the HS test.

Figure 8 Concrete damage in the major and minor direction of loading (a) lower part (b) upper part

To restore the damaged RC#1 column deformation capacity and investigate its seismic response while enduring further ground motion effect using HS technique, a specially-designed strengthening program was planned and followed based on the specimen's resulted damage. Consequently, to improve the seismic performance of the damaged column after being subjected to four different levels of orthogonal earthquake records, transverse layers of CFRP were used and a rapid retrofitting process was carried out within few days. This process started with maintaining an axial loading ratio of about 4.7%; equal to 56% of the original column loading to replicate the original applied gravity load. Then all the crushed and damaged concrete was carefully removed to prepare the specimen surface for closing and sealing all the observed flexural cracks concentrated at both specimen's ends using a high-performance vinyl ester resin FIS V 360 S.

A cementitious polymer bonding primer was applied, Sika® MonoTop®-910N over the concrete cracks. Replacing the damaged concrete took under consideration using a repair mortar Sika® MonoTop®-412N of similar compressive strength value of the damaged specimen; equal to 40MPa (Figure 9).

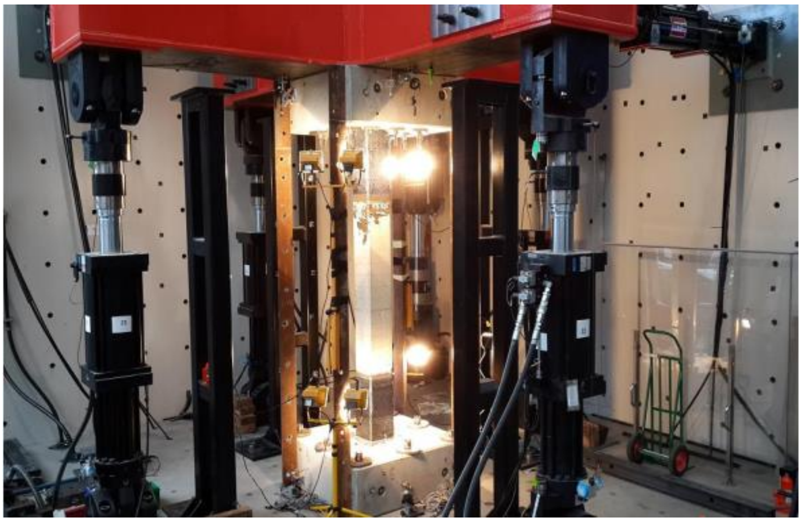


Figure 9 Damaged column specimen retrofitting program at SSL, Swinburne

The CFRP retrofitting process included applying unidirectional carbon fibers in the transverse direction; some of the fibers properties were provided by the manufacturer such as the ultimate tensile strength 4900MPa, tensile modulus 230GPa. This process was followed by carrying out a curing process as illustrated in Figure 9 which shows the retrofitting program as well.

RC# 2 COLUM EXPERIMENTAL TEST RESULTS

RC#2 column test results included a detailed comparison with RC#1 column specimen in term of mode of failure, the hysteretic lateral force-drift ratio relationship, hysteretic dissipated energy, stiffness degradation, lateral force- drift and lateral force-curvature relationships. It should be noted that the retrofitted RC#2 column was subjected to an additional scale level of the Imperial Valley biaxial ground motion records; 7 (g), added to cover the increase in the column deformation capacity that resulted from wrapping the damaged specimen with CFRP sheets (Lam and Tang, 2003& ACI440.2R, 2008).

Accordingly, RC#2 was tested to structural OMRF building structural failure which showed similar performance levels with CFRP horizontal shear tearing at Life Safety Level Earthquake (LLE), Collapse Level Earthquake (CLE) and a Final Collapse Level Earthquake (FCLE) which was corresponded to 3.5%, 6.42% and 8.09% drift ratios particularly in the major direction of loading. The concentrated damaged zones at both ends of RC#2 column demonstrated a much more controlled and uniform failure compared to RC#1 located at 200mm from both ends, therefore showcasing the effect of CFRP wrapping. Where the inspection process that followed conducting HS test showed CFRP sheets rapture and debonding from the specimen's surface in both loading directions associated with horizontal cracks and parallel concrete surface cracking. In addition, the CFRP sheet confinement prevented transverse and main reinforcement fracture although buckling of steel bars towards the RC#2 corners was much more noticed (Figure 10).

Figure 10 RC#2 column a) CFRP rupture b) lower end and (c) upper end both in the major direction

By comparing the experimental behavior of RC#2 column to RC#1 column a similar asymmetric hysteretic performance was noticed as can be shown in Figure 11.

However, a more ductile response was observed for RC#2 with a curvature ductility (μϕ) equal to 9.5 compared to 6.1 for RC#1 in the major direction of loading. Furthermore, a smaller lateral strength capacity reduction between RC#2 and RC#1 was observed in the minor direction of loading with 14% compared to 28% reduction in the lateral strength in the major direction.

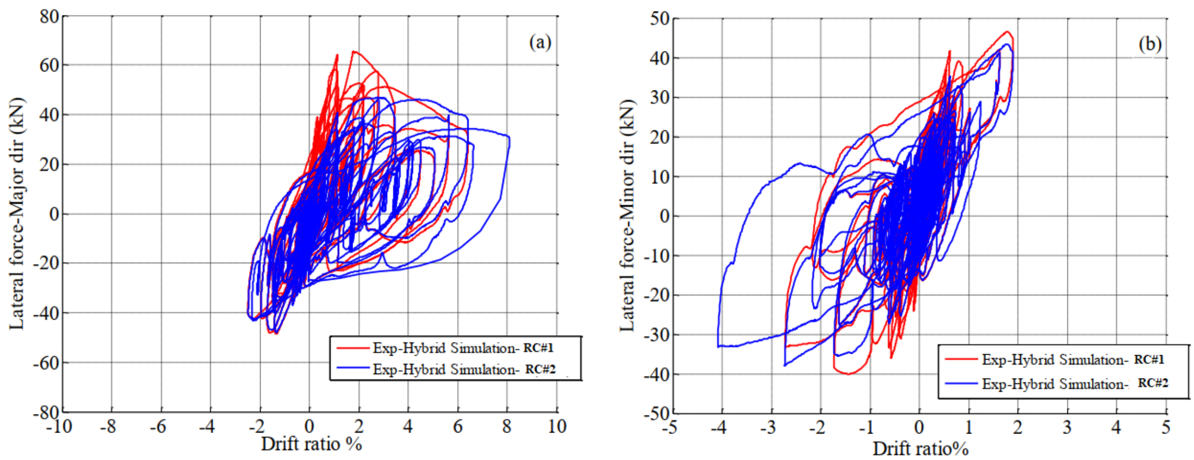
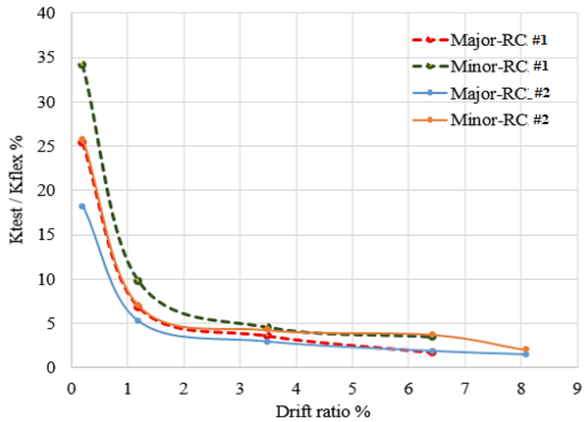
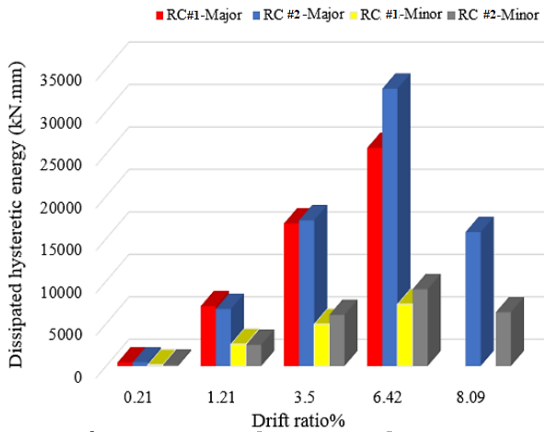


Figure 11 Comparison of hysteretic lateral forces-drift ratio relationships for RC#1 and RC#2 columns



RC# 1 and RC#2 columns



for RC# 1 and RC#2 columns

Figure 12. Stiffness degradation and dissipated energy relationships

The stiffness degradation of both RC#1 and RC#2 columns in Figure 12 (a) clearly illustrated a similar parabolic reduction, although RC#2 maintained its stiffness ratios at drift larger than 3.5%. The effect of CFRP sheet confinement was clear when approaching the end of HS test as RC#2 column reached similar stiffness values to RC#1 between both directions of loading at 8.09%drift despite the state of damage that was affecting the stiffness values at lower drift ratios. In addition, similar performance can be noticed in Figure 12 (b) for both RC#1 and RC#2 columns by comparing their energy dissipation response for small drift ratios and drift ratios larger than 3.5% demonstrating the effectiveness of CFRF wrapping on the biaxial seismic performance of RC#2 while enduring higher levels of earthquake ground motion.

CONCLUSION

The effectiveness of using CFRP sheets was demonstrated in this paper as an efficient method for restoring the lateral deformation capacity of RC columns in frame buildings subjected to real earthquake records through especially designed retrofitting program described as a rapid strengthening framework that can be applied to damaged RC frame columns. The successfulness of this framework in simulating the collapse of RC structurers is attributed to the HS testing method that allowed a 6-DOF control between numerical and experimental subassemblies of large RC components that was carried out by the state-of-art, MAST system in Swinburne University of Technology, Melbourne, Australia. Consequently, the experimental test results showed more-pronounced increase in RC#2 deformation capacity while enduring higher level of biaxial earthquake demands that approached drift value of 8.09% compared to 6.42% for RC#1. Furthermore, the retrofitted column RC#2 approximately maintained its stiffness degradation, hysteretic dissipated energy and lateral strength capacity in large drift ratios particularly in the minor direction of loading.

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