SEISMIC BAHVIOUR OF FRP REINFORCED CONCRETE FRAME BUILDINGS

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ABSTRACT :

The use of fibre reinforced polymer (FRP) reinforcement in buildings is gaining acceptance in the construction industry due to their superior corrosion resistance, durability and higher strength. Of the various types of fibres used in producing FRP reinforcement, carbon fibers provide higher elastic modulus. Therefore, carbon-fibre reinforced polymer (CFRP) bars and grids are more suitable for use in building construction. Research on seismic performance of FRP reinforced concrete buildings is currently limited worldwide.

The paper reports on dynamic response of two CFRP reinforced concrete buildings designed for Vancouver, Canada, following the seismic requirements of the National Building Code of Canada (NBCC 2005) and the Canadian Standards Association (CSA) S806-02 (2002) for “Design and Construction of Building Components with Fibre-Reinforced Polymers”. A computer program was developed for dynamic response history analysis, incorporating a hysteretic model for FRP reinforced concrete elements. The buildings were analyzed under NBCC compatible earthquake records to establish design force and deformation demands. The results indicate that FRP reinforcement can be used to reinforce concrete buildings in seismically active regions with structural elements designed not to suffer from the rupturing of CFRP prior to the onset of concrete crushing. FRP reinforced concrete buildings designed to respond in the elastic mode of deformations remain within the force and deformation demands indicated in building codes. Inelastic response of buildings under amplified ground excitations indicate that it is possible to reduce design force levels through limited inelasticity provided by confining the compression concrete in members and eliminating tension failure in FRP reinforcement.

KEYWORDS: concrete buildings, confinement, drift capacity, ductility, dynamic response, fiber reinforced polymers, FRP, hysteretic behavior, seismic design, seismic analysis.

1. INTRODUCTION

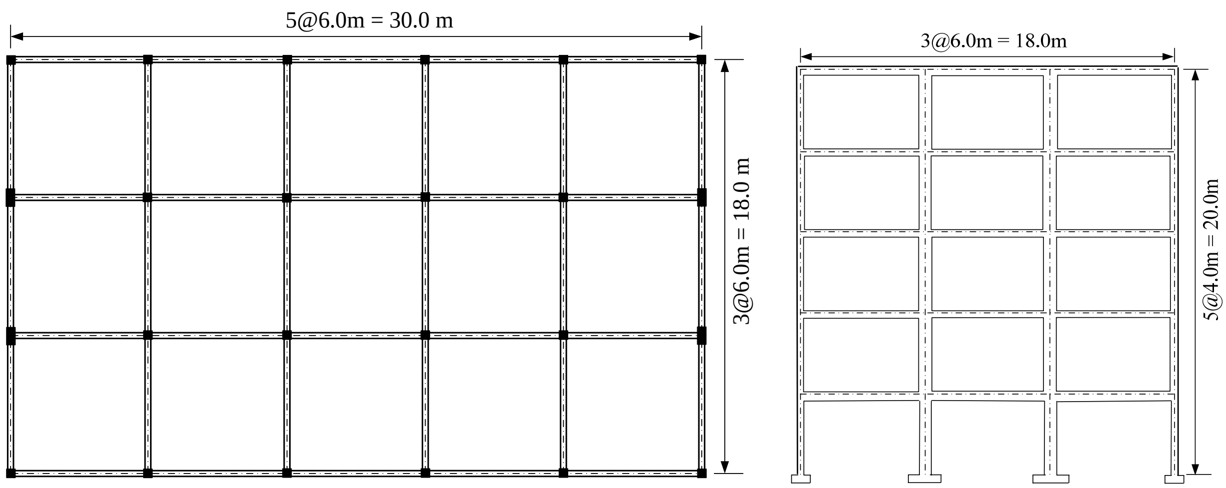
The use of fiber reinforced polymers (FRP) as a construction material has increased in recent years, primarily because of the non-corrosive nature and high tensile strength of the material. Though the principle application of FRPs has been in the form of glass and carbon sheets for retrofit and rehabilitation projects, FRP reinforcing bars are being considered as an alternative to steel reinforcements for use in new reinforced concrete structures. A major challenge for using FRP re-bars in seismically active regions remains to be their brittle failure characteristics.

Extensive experimental research has been conducted in the Structures Laboratory of the University of Ottawa to investigate the seismic performance of FRP reinforced concrete beams and columns. A hysteretic model was developed for dynamic response history analysis. The computer program SEQUAKE was developed, incorporating the hysteretic model.

The current investigation, reported in this paper, involves dynamic inelastic response history analyses of selected FRP reinforced concrete frame buildings. Frame buildings with 5-storey and 10-storey heights were designed for Vancouver, BC in Canada, following the requirements of the Canadian Standards Association Standard S806-02 (CSA 2002). The seismic response of buildings was investigated through response time history analysis under synthetic earthquake records that reflect the seismicity of the region and are compatible with the Uniform Hazard Spectrum (UHS) for the region defined in the National Building Code of Canada (NBCC 2005). The response, in terms of maximum storey drift and base shear force demands, is presented in the paper. Inelastic response characteristics are presented in terms of recoded hysteretic relationships. Design information is presented based on the results of analyses.

2. SELECTION AND DESIGN OF BUILDINGS

Two FRP reinforced-concrete frame buildings, with 5 and 10 storey heights and a symmetric rectangular floor plan, were designed for the city of Vancouver in Canada. The floor plan and elevation view of the 5-storey building are shown in Figure 1. The 10-storey building had the same storey height and the same floor plan except for the use of rectangular columns as interior columns. The cross-sectional details of beams and columns are summarized in Figure 2.

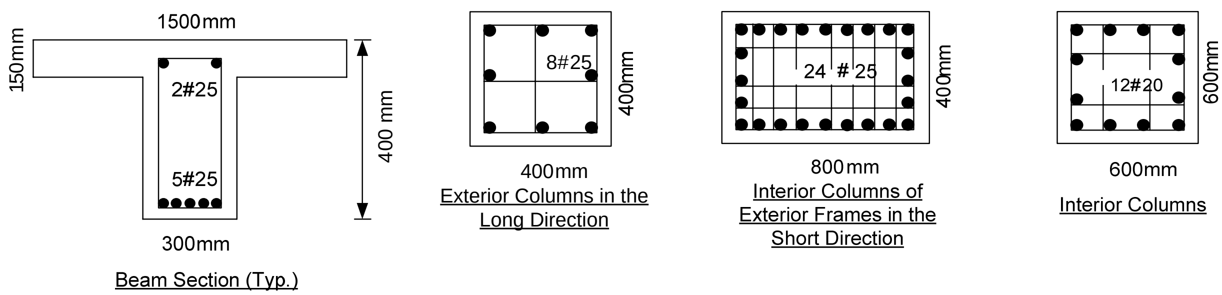


a) Plan view b) Elevation view in the short direction

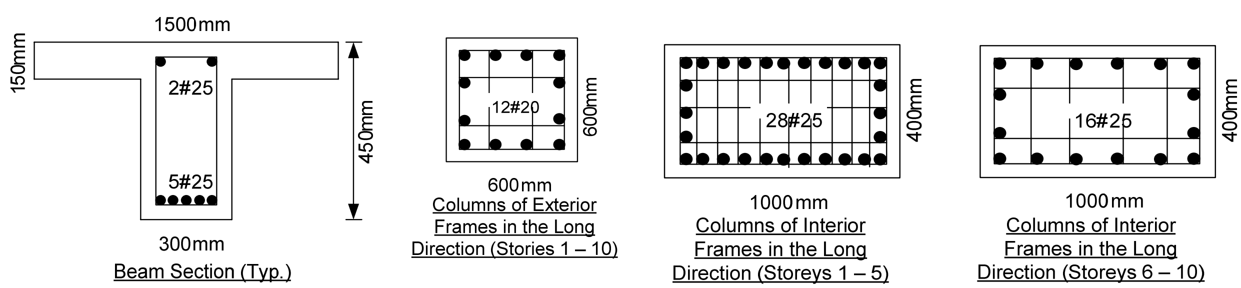
Figure 1 Geometric details of the 5-storey building designed for Vancouver

The structures were designed following the requirements of NBCC-2005. Dead and live loads were selected as 5.0kN/m^2 and 2.4kN/m^2 for floors and 3.5kN/m^2 and 2.2kN/m^2 for roofs, respectively. Normal strength concrete, with f'_c= 40MPa was used throughout the design. The FRP reinforcement consisted of sand coated Carbon-FRP (CFRP)_bars with a nominal bar diameter of either 19.5 mm (#20; Area = 200 mm^2) or 25.2 mm (#25; Area = 500 mm^2). The tensile rupturing strength of FRP was 1596 MPa as established through coupon tests. The columns and beams were confined with FRP grids to ensure inelastic deformability. CSA Standard S806-2002 was employed for proportioning and detailing of individual elements. Static analyses required for design were carried out using the computer program SEQUAKE details of which are provided in the next section. Equivalent static seismic forces were obtained from NBCC-2005 and the UHS specified in the code for the city of Vancouver. The static analyses provided design values for axial forces, shear forces, bending moments and interstorey drifts.

The structural elements were designed to have over-reinforced sections to prevent failures by FRP bar rupturing. This was felt essential because the buildings were designed to sustain significant earthquake forces with potential inelasticity developing in their critical regions. Therefore, the area of longitudinal reinforcement was increased to promote concrete crushing at failure prior to the rupturing of FRP reinforcement. This created challenges for T-beam design as these beams typically have large concrete compression zones, requiring very large areas of FRP reinforcement.



a) 5-Storey building



b) 10-Storey building

Figure 2 Cross sectional details of elements for building selected and designed

Columns were first designed to have square sections. However, this resulted in lateral drift ratios higher than the limit specified in the code. According to NBCC-2005, the interstorey drift is limited to $0.025h_s$ for ordinary buildings and $0.02h_s$ for post-disaster and school buildings, where h_s represents the storey height. Therefore, the column sizes were increased and more rigid rectangular sections were used as shown in Fig. 2 to meet the drift limit. The resulting frames had fundamental periods of 1.47 sec and 2.25 sec for 5 and 10-storey buildings, respectively. The fundamental periods computed on the basis of the empirical expression given in NBCC-2005 for reinforced concrete buildings produced 0.71 sec and 1.19 sec for 5 and 10-storey buildings, respectively.

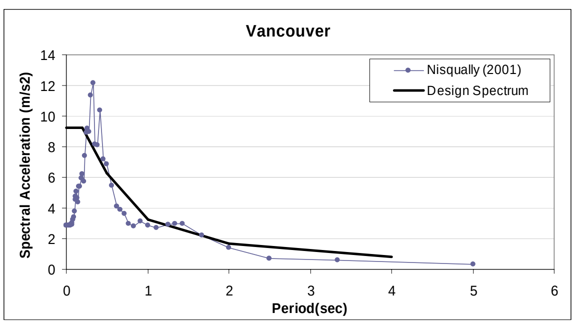
3. SELECTION OF GROUND MOTION RECORDS

One of the primary objectives of the current investigation was to assess the seismic performance of FRP reinforced concrete frame buildings in western Canada. Therefore, it was important to select ground motion records which were compatible with the UHS specified in NBCC-2005. NBCC-2005 defines UHS by four site-specific spectral acceleration values based on 5% damping at periods of 0.2, 05, 1.0 and 2.0 seconds. The UHS was developed to provide maximum design spectral acceleration values and was generated by considering a range of earthquakes that contribute most strongly to the hazard at a specified probability level for the period values mentioned above. Consequently, the curve passing through these four spectral values constructs the idealized UHS for each location with uniform hazard across the country. The probability level used in deriving UHS was 2% in 50 years.

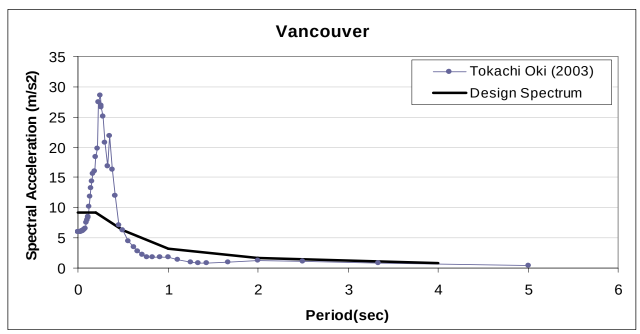
Two different types of ground motion records were selected for seismic analysis. The first type was artificially generated by Atkinson and Beresnev (1998). The researchers generated eight horizontal components for Vancouver such that four records represented a moderate earthquake nearby and the other four represented a stronger earthquake farther away. This was found necessary to mach the moderate and strong distant earthquakes to short and long period energy bands. The eight artificial ground motion records generated were based on moment magnitudes of M6.5 and M7.2 for short and long period hazard with hypocentral distances of 30 and 70 km, respectively. The resulting time histories were then scaled up or down by a 'fine tuning factor' in order to mach the target spectra as much as possible.

The second type of earthquake records used in the current study was those recorded previously during actual earthquakes. In order to scale these records to match with the UHS, elastic response spectra for the records were plotted. These spectra were then compared with the design spectrum for Vancouver.

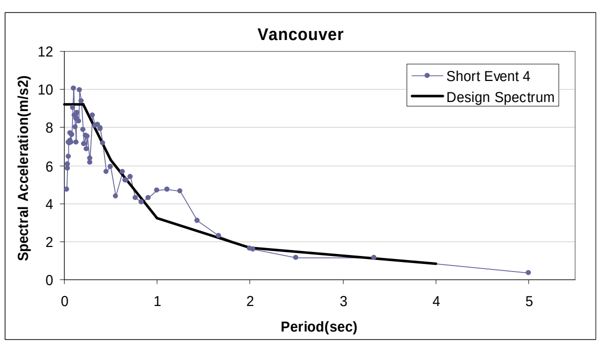
The ground motion records with spectral values closest to UHS for the period range of structures considered were selected for time history analysis. Among the artificially generated 8 records, the Short Event No: 4 was found to govern response within the period range for 5 and 15-storey buildings. Among the previously recorded earthquakes, the 2001 Nisqually and 2003 Tokachi Oki records showed closest matches with the UHS for 5 and 10-storey buildings, respectively. Figure 3 shows the comparisons of response spectra of these earthquake records with UHS for the city of Vancouver.



(a) Response spectrum for Nisqually 2001 Record



(b) Response spectrum for Tokachi Oki 2003 Record

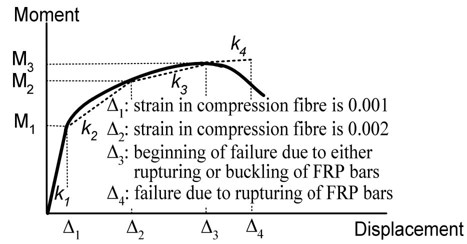


(c) Response spectrum for Short Event No: 4

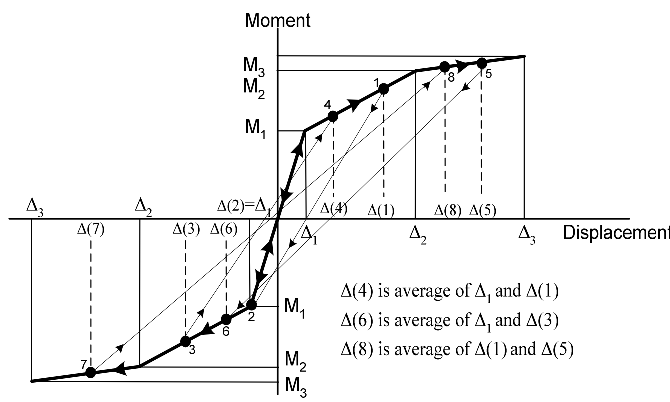
Figure 3 Comparisons of response spectra of selected earthquake records with UHS for Vancouver

4. COMPUTER SOFTWARE "SEQUAKE"

Computer program, SEQUAKE (program for **S**tructural analysis under **E**arth**QUAKE** loading) was developed for analysis of three-dimensional nonlinear static and dynamic response of reinforced concrete structures. The program utilizes the stiffness method of analysis with frame members idealized as beam-column elements. Each element has flexural springs at the ends to simulate plastic hinges. The changes in stiffness matrix of an element are taken into account by considering the tangent stiffness on the moment-rotation relationship of springs. The moment-rotation relationships of end springs are obtained by incorporating hysteretic models developed for flexural behaviour of FRP and steel reinforced concrete members. For steel reinforced elements, the hysteretic model of Clough (1966) was adopted. For FRP reinforced elements, the hysteretic model of Shahbatdar and Saatcioglu (2003) was incorporated, as shown in Figure 4. Dynamic response is determined for each time increment (time step) using the time-integration method. Linear structural behaviour is assumed during each time step.



a) Primary curve



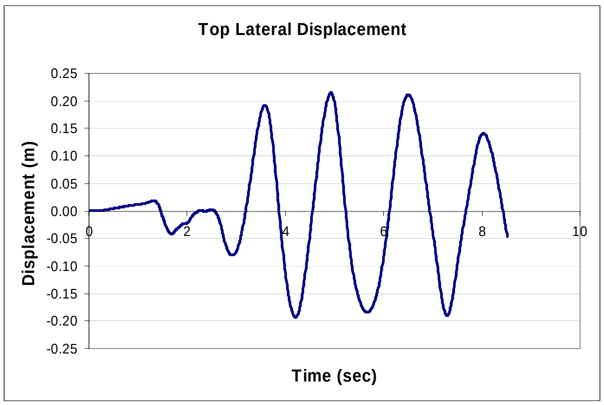
b) Rules defining unloading and reloading

Figure 4 Hysteretic model developed by Sharbatdar and Saatcioglu (2003) for FRP reinforced concrete elements

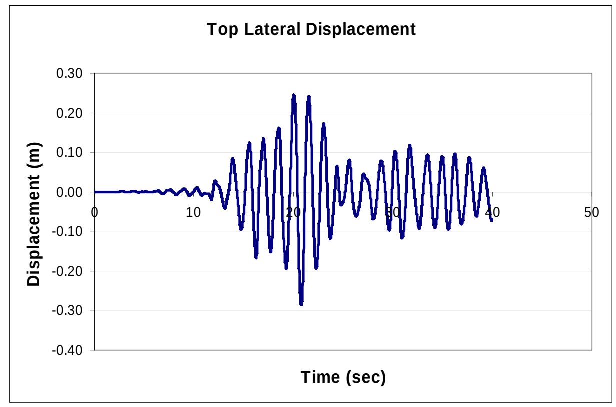
5. DYNAMIC RESPONSEN OF FRP REINFORCED CONCRETE BUILDINGS

Two sets of analysis were conducted for each building under artificially generated and previously recorded earthquake ground motions, scaled to match UHS for Vancouver. The buildings were analyzed in the short direction of floor plan. Two external and four internal frames were lumped together and linked with rigid links to simulate rigid floor response, creating equal displacements of frames at each storey level. The softening of individual elements due to the penetration of yielding into the adjacent elements and the elongation of longitudinal reinforcement within the attached member (referred to as anchorage slip) was modeled as percentage reduction in flexural rigidity, when this option was considered. However, the analysis results presented in this paper neglected the anchorage slip.

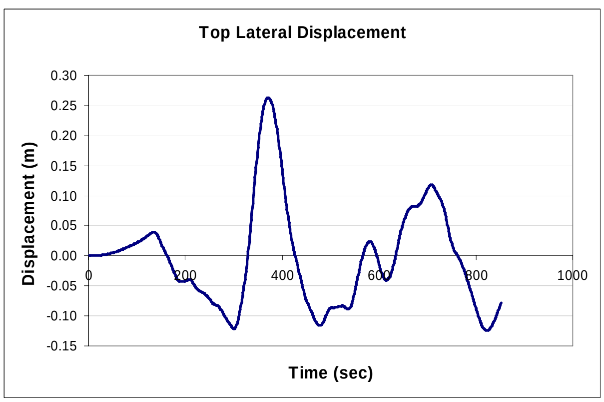
The analysis results indicated that the buildings remained elastic under design earthquake forces; with maximum storey drifts limited to 1.4% and 1.9% for the 5-storey building and 1.0% and 0.6% for the 10-storey building under artificially generated and previously recorded ground motions, respectively. Figure 5 illustrates response time histories for the roof of each building. The buildings were initially designed to respond elastically because of the uncertainties associated with nonlinear performance of FRP reinforced concrete buildings. Therefore, elastic response during dynamic analysis was expected. The comparisons of seismic base shear demands indicated that the design base shear force of 4950 kN for the 5-storey building obtained from equivalent static load analysis was reduced to 3478 kN and 4370 kN under Short Event No: 4 and Nisqually 2001 records. Similarly, the static design base shear force of 6572 kN was reduced to 3443 kN and 2640 kN when subjected to Short Event No: 4 and Tokachi Oki 2003 records. These comparisons indicate that even under the UHS compatible earthquake records, a significant reduction may be obtained in base shear demands when computed through dynamic analysis. This may be attributed to the interaction of dynamic characteristics of structures with those of the exciting forces, whereas the elastic design forces are based on empirically obtained periods, which tend to be substantially shorter than those computed through dynamic analysis.



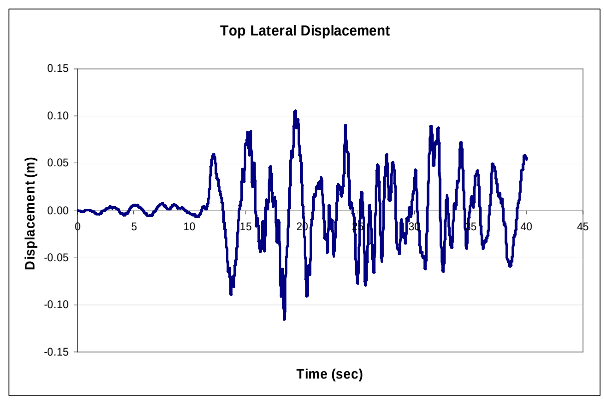
a) 5-Storey building under Short Event No:4



b) 5-Storey building under Nisqually (2001) record



c) 10-Storey building under Short Event No:4

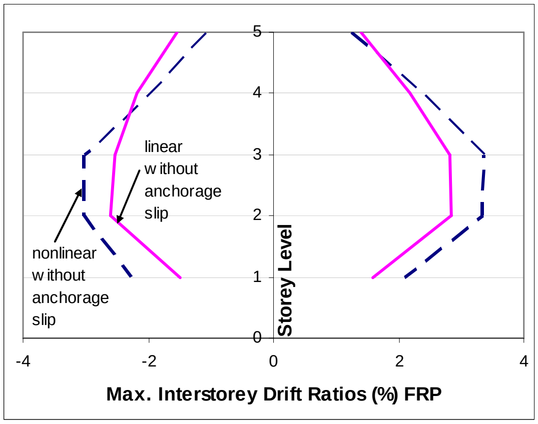


d) 10-Storey building under Tokachi Oki (2003) record

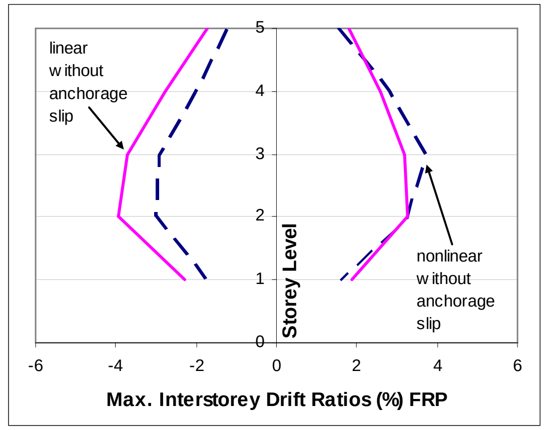
Figure 5 Displacement time histories for buildings at the roof level

Inelastic response of FRP reinforced concrete buildings was investigated by analyzing the 5-storey building under intensified earthquake records. This was done by amplifying the previously selected earthquake records by a factor of 2.0. Each analysis was conducted twice, with and without inelasticity permitted in critical regions of members. Nonlinearity resulted in 18% to 50% reduction in based shear under intensified 'Short Event 4' and the 2001 Nisqually earthquake record. The maximum interstorey drift ratios either increased by 38% to 91% or remained approximately the same with the consideration of inelasticity in analysis. Though drift demands generally increased with inelasticity, there was no consistent pattern observed. The structural response changed considerably by the consideration of inelasticity in response because of the changes in dynamic characteristics of buildings and the interaction with the frequency contents of exciting ground motions.

Maximum interstorey drift ratios with and without the consideration of inelasticity are compared in Figure 6. Because of the increased intensity of earthquake motions, the maximum interstorey drift limit of 2.5% specified in NBCC-2005 was exceeded.



a) Under Short Event No: 4



b) Under Nisqually 2001 earthquake record

Figure 6 Comparisons of storey drifts, with and without inelasticity

The significance of inelasticity on member response was investigated by examining the hysteretic characteristics of FRP reinforced concrete elements. Sample hysteretic relationships, obtained from analyses are shown in Figures 7 and 8. Some of the structural elements experienced failure when the intensified earthquake records were used. The intensified Short Event No: 4 resulted in the failure of all beams on the first three stories through the crushing of concrete which was followed by the rupturing of FRP. Some of the first storey columns also failed due to the crushing of concrete. The intensified 2001 Nisqually record resulted in the crushing of concrete and subsequent rupturing of FRP in all beams on the first three stories, as well as the columns on the first storey. The buildings experienced a small reduction in force demands due to the inelasticity of columns while maintaining approximately the same deformation demands. The results indicate that well confined columns could develop some limited ductility even though the beams experienced brittle failures beyond their elastic limits without any significant inelastic deformability.

6. CONCLUSIONS

The following conclusions can be drawn from the analytical investigation reported in this paper:

- CFRP bars and grids can be used as concrete reinforcement. CFRP reinforced concrete buildings can be designed for seismically active regions with due considerations given to the failure mode and inelastic deformability of members. Because of the brittle characteristics of FRP reinforcement, the sections should be designed to be over-reinforced with failures initiating through the crushing of concrete. Confinement of concrete can promote inelastic deformability in FRP reinforced concrete buildings.
- Seismic force demands established through dynamic analysis can be lower than those computed based on equivalent static load analysis with empirically determined period values.
- In spite of the lower elastic modulus of CFRP reinforcement, lateral drift demands established through dynamic analysis can be within the drift limits established in building codes.
- Effect of inelastic response is to reduce seismic force demands with some increase in drift demands.

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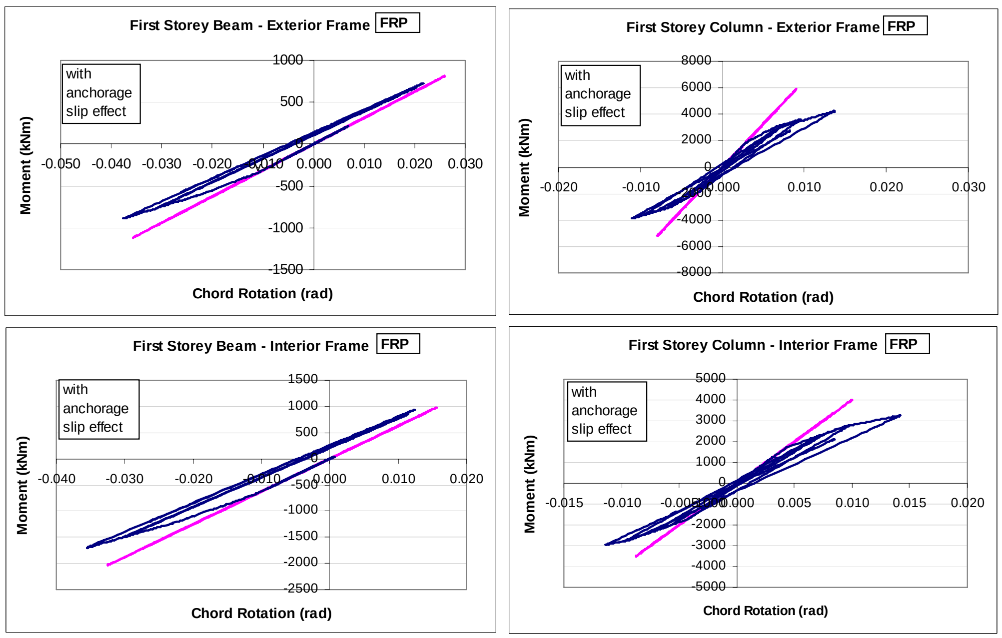

Figure 7 Selected hysteretic relationships under Short Event No: 4

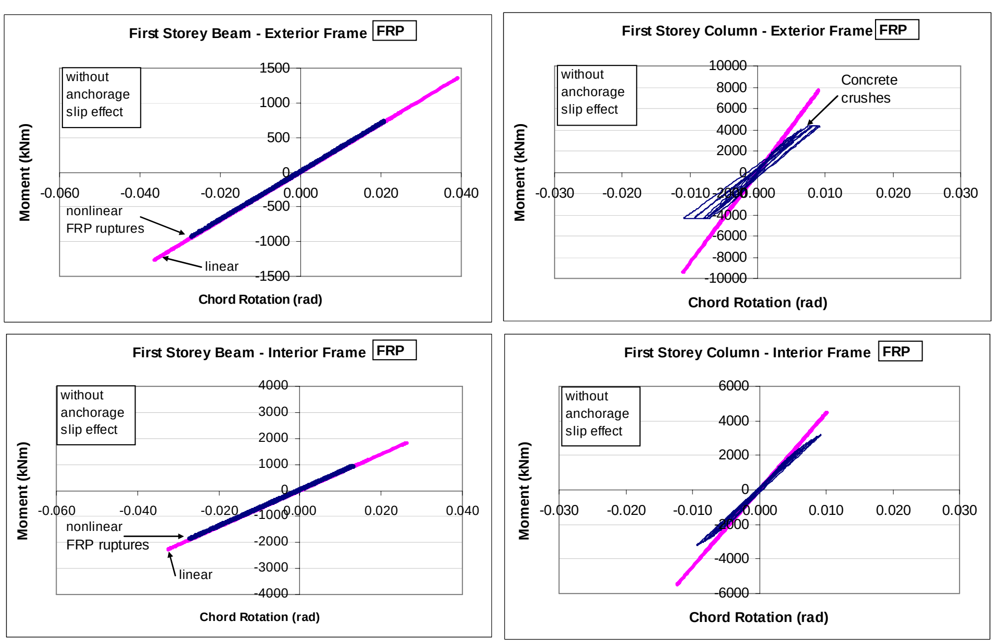

Figure 8 Selected hysteretic relationships under the 2001 Nisqually record