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SEISMIC RESPONSE OF FRP REINFORCED CONCRETE BUILDINGS

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ABSTRACT

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 reinforcement 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.

An analytical study has been carried out by using a computer program developed for nonlinear analysis of steel and FRP reinforced concrete structures. The results indicate that force and ductility demands of FRP reinforced concrete frames may be reduced because of the reduced stiffness and longer periods associated with low elastic modulus FRP reinforcement. The results further indicate that seismic design of FRP reinforced concrete structures should be viewed differently than steel reinforced concrete structures with the objective of attaining sufficient deformability and drift capacity while promoting primarily elastic response with limited ductility demands.

Introduction

The application of FRPs in the construction industry has been dominated by the rehabilitation of older structures experiencing corrosion related durability problems and/or retrofitting seismically deficient structures by externally applied FRP sheets. The application of FRPs to new structural designs has been limited because of insufficient knowledge in the field and lack of experience and confidence in short and long-term performance of FRP reinforced concrete structures. The majority of field applications for new construction have been for bridge decks (Nanni 2000, Uomoto et al. 2002, Bakis et al. 2002, Humar and Razaqpur 2000), with additional examples involving non-structural elements (ex: concrete medium barriers) and portions of buildings that require electromagnetic neutrality, as in the case of specialized hospital facilities. Additional examples of the use of FRP rebars include the Laurier-Taché parking garage in Hull-Quebec (Benmokrane et al. 2004) and the MRI unit of a hospital building in San Antonio-USA (ACI 2001).

Several standards and design guidelines have been developed in recent years for concrete structures reinforced with FRP bars, including ACI 440.1R-03 (ACI 2003), CSA Standard S806-02 (CAS 2002) and

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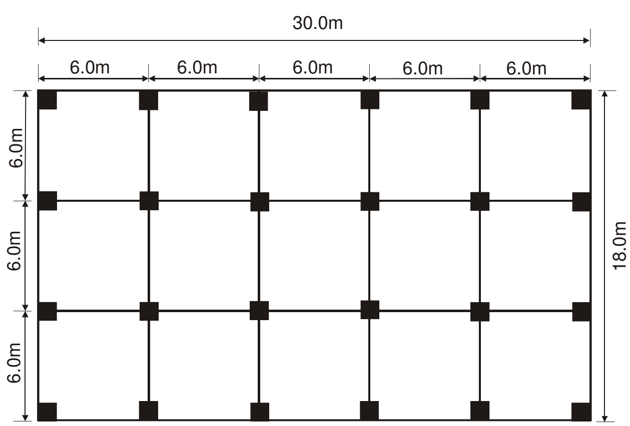
ISIS-M04-00 (ISIS Canada 2000). While these documents reflect the state-of-the art at the time of their publication, they lack complete design information, especially in the area of seismic design. Indeed, the performance of FRP reinforced concrete structures under seismic loading has not been well researched. There is lack of test data on performance of FRP reinforced concrete elements under inelastic deformation reversals. One area that specifically lacks critical information is the performance of FRP reinforced concrete columns and beams where the FRP bars experience tension-compression cycles while concrete develops inelastic deformations. Several studies were conducted on columns (Kobayashi and Fujisaki 1995; Daniali and Paramanantham 1994; Alsayed et al. 1999; Saatcioglu and Sharbatdar 2000; and Fukuyama et al. 1995) though the information generated by these recent research programs has not yet been incorporated in design standards and guidelines.

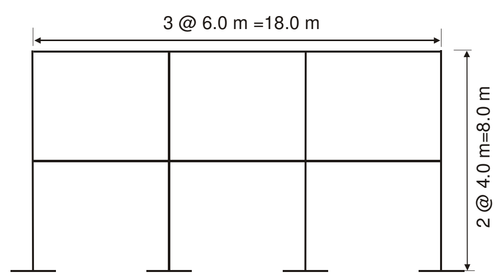
An extensive experimental and analytical research is underway at the University of Ottawa to investigate the seismic performance of FRP reinforced concrete beams, columns and overall structural systems. The research program resulted in a hysteretic model for FRP reinforced concrete elements (Sharbatdar and Saatcioglu 2007) and a computer program incorporating the model by the authors. The current research project, reported in this paper involves dynamic inelastic response history analyses of FRP reinforced concrete frame buildings subjected to ground motion records compatible with the uniform hazard spectra of the National Building Code of Canada (NBCC 2005). Two-storey, low-rise frame buildings were designed for Ottawa and Vancouver, reflecting the seismicity of eastern and western Canada, on the basis of the requirements of CSA Standard S806-02. The buildings were subjected to NBCC compatible artificial ground motion records. Their strength and deformability were assessed and compared with those of companion steel reinforced concrete buildings.

Selection of Structures and Seismic Records

A 2-storey moment resisting frame building was selected and designed for Ottawa, representing a medium seismic region and for Vancouver, representing a region of high seismicity. The designs were carried out as; i) a steel reinforced concrete building, and ii) an FRP reinforced concrete building. The plan view and the elevation of the building are illustrated in Fig.1.



(a) Floor Plan



(b) Elevation

Figure 1. Moment resisting frame building selected for analysis.

A symmetrical floor plan was selected to minimize the effects of torsion. The design of the steel reinforced structure was carried out following the requirements of the CSA Standard A23.3 (2004), while the FRP reinforced concrete building was designed using the CSA Standard S806-02 and ISIS-M04-00. Normal-strength concrete, with f'_c = 30 MPa was used throughout the design. The longitudinal steel reinforcement was of Grade 400 MPa #20 deformed bars with a nominal area of 300 mm^2, while the FRP reinforcement was of sand coated #15 carbon-FRP bars with a nominal area of 200 mm^2 and a tensile strength of 1596

MPa. The columns and beams were confined with steel ties and hoops in the case of steel reinforced buildings and FRP grids in the case of FRP reinforced buildings, to ensure inelastic deformability of concrete.

The structures were analysed with gravity and static seismic loads for design. The design base shears were calculated according to the equivalent static load procedure of NBCC 2005. Accordingly;

$$V = \frac{S(T_a)M_v I_e W}{R_o R_d} \tag{1}$$

where, $S(T_a)$ is the design spectral acceleration for fundamental period T_a; M_v and I_e are factors reflecting higher mode effects and building importance, both taken as 1.0. “W” is the total weight of structure. The elastic base shear for steel reinforced concrete buildings is reduced by the product of the ductility and overstrength reduction factors, R_d and R_o, respectively. The R_d factor was taken as 2.0 for Ottawa and 4.0 for Vancouver, corresponding to nominally and fully ductile structures; while the overstrength factor was selected as 1.7 for buildings in both cities. The FRP reinforced buildings were designed for elastic seismic forces with R_d = R_o = 1.0. The design details of steel and FRP reinforced structures are provided in Fig.2.

Four NBCC design-spectra compatible ground motion records, synthetically generated by Atkinson and Beresnev (1998) were selected for dynamic analysis. One record for each location provided compatibility with design spectrum in the short period range while another record was compatible in the long-period range. Therefore, a total of four records were used, two for Ottawa and two for Vancouver.

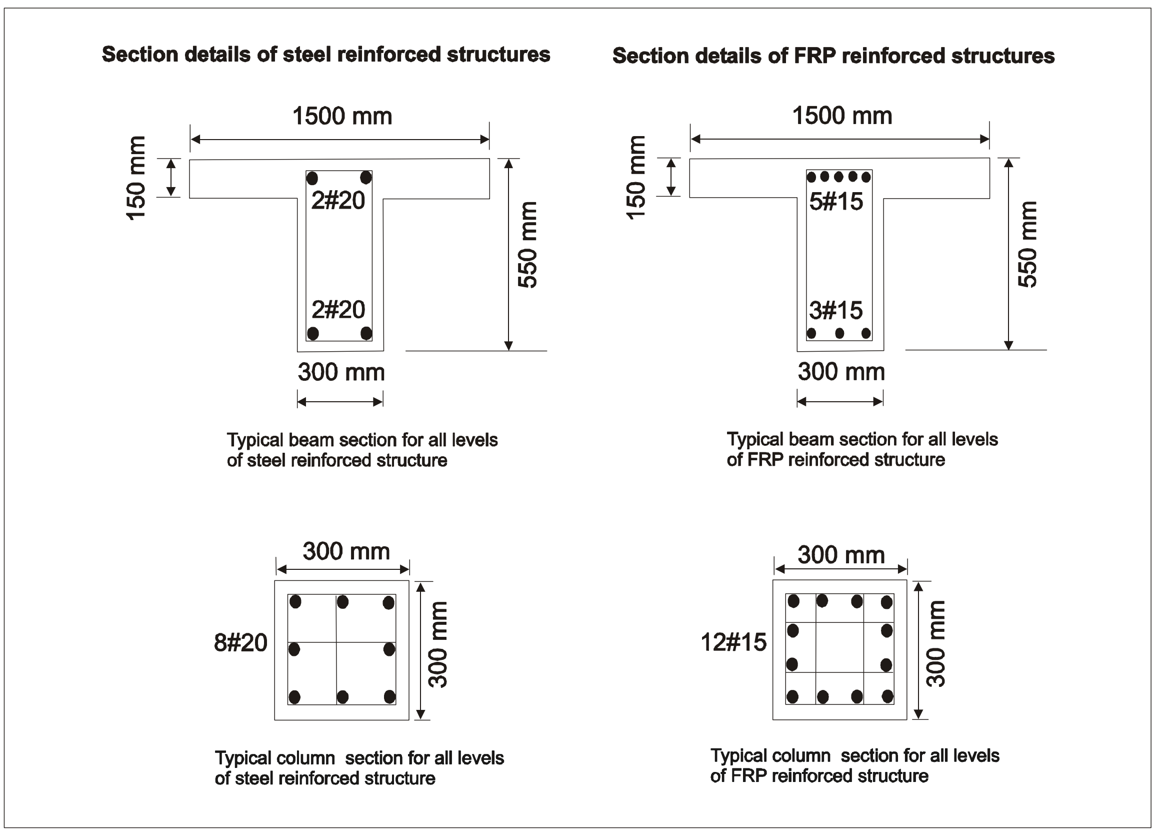


Figure 2. Design details of frame elements for steel and FRP reinforced structures.

Dynamic Analysis Procedure and Computer Software

A computer program was developed for dynamic inelastic response history analysis of buildings. The program employed the step-by-step integration technique to solve the equation of motion for each time increment *Δt*. The equation of motion for a given time increment of *Δt* is shown below.

$$\Delta R_t = M \Delta \ddot{U}_t + C \Delta \dot{U}_t + K \Delta U_t \tag{2}$$

Where, ΔR_t is the incremental load vector due to earthquake *M*, *C* and *K* are the mass, damping and stiffness matrices respectively. The stiffness matrix trances continuous changes in stiffness during inelastic response and accounts for any change in stiffness in a given time step. The changes in stiffness due to inelastic behaviour are incorporated by following the rules of hysteretic models. The assemblage procedures for mass, damping and stiffness matrices are adopted from Erkmen (2001). The symbols; $\Delta \ddot{U}_t$, $\Delta \dot{U}_t$ and ΔU_t represent the incremental acceleration, velocity and displacement vectors at time *t*. For the solution of the equilibrium equation at time *t*, the acceleration, velocity and displacement vectors of the previous time step are used.

Inelasticity in a member is introduced through a flexural spring at each end as illustrated in Fig. 3. This implies that inelasticity is considered only for the flexural component of deformations. This is justified because of the flexure dominant response displayed by frame elements.

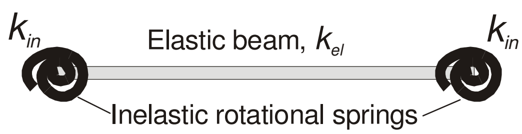


Figure 3. Elastic beam with nonlinear rotational springs for flexure.

Hysteretic behaviour of members is introduced through hysteretic models. The hysteretic model developed by Takeda et al. (1970) is adopted for steel reinforced concrete members. Fig. 4 illustrates the general features of Takeda's model which reflect stiffness degradation of reinforced concrete under reversed cyclic loading. The model consists of a primary force-deformation relationship (backbone curve) computed from geometric and material properties of members and a set of rules that define stiffness changes during unloading and reloading phases of reversed cyclic loading. This is done by computing the stiffnesses of the inelastic springs, k_{in} based on the slopes k_1 and k_2 of the primary force-deformation relationship. Unloading is assumed to occur following the same stiffness as the initial effective elastic stiffness (which incorporates softening due to cracking) and reloading stiffness is found such that the force-deformation relationship aims at the previous maximum deformation point on the primary curve. Therefore, the reloading stiffness gets softer with increasing maximum deformation.

Stiffness changes in FRP reinforced members under reversed cyclic loading are simulated using the hysteretic model developed by Sharbatdar and Saatcioglu (2007). The hysteretic model consists of a primary moment-flexural displacement relationship that defines the strength boundary and initial stiffnesses during initial loading, and a set of rules defining the variations in stiffness during unloading and reloading. The primary curve has a tri-linear relationship as illustrated in Fig. 5(a). The first segment represents the elastic stiffness up to a strain of $\varepsilon = \varepsilon_0 / 2$ at the extreme compression fiber of critical section, where ε_0 is the concrete strain at peak compressive strength f'_c and may be taken as 0.002 for normal-strength concrete. The second segment represents cracked stiffness up to the beginning of cover spalling at a maximum compressive fiber strain equal to ε_0. After the cover starts spalling off, there is a significant change in slope. The third segment is the portion between the cover spalling point and the peak failure point on the primary curve corresponding to the failure of FRP reinforcement. The failure of FRP may be triggered either by local buckling of fibres in FRP bars or by rupturing in tension.

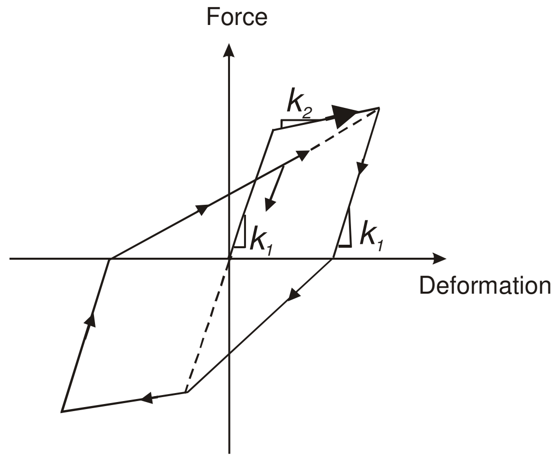
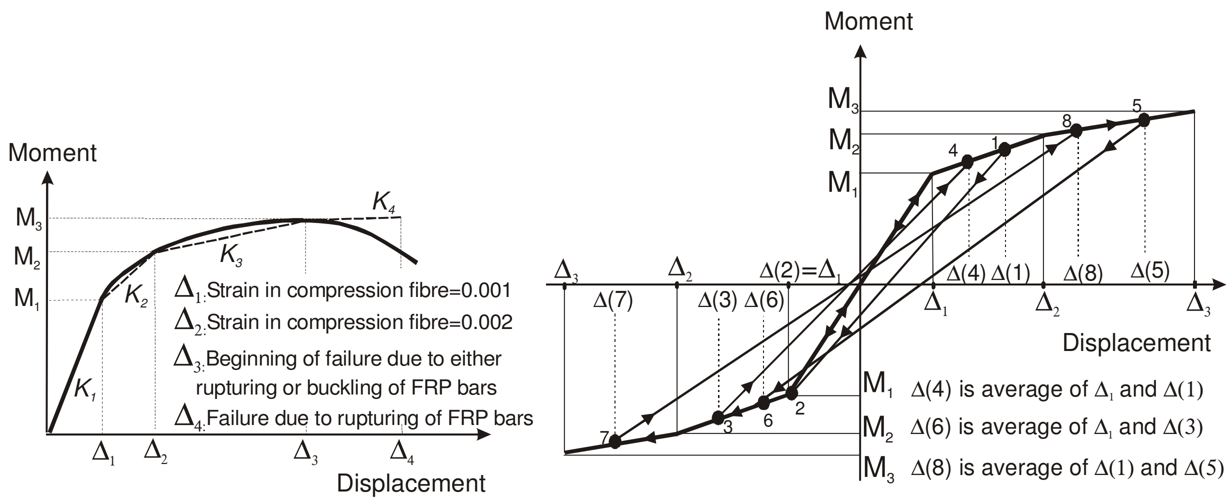


Figure 4. Hysteretic model by Takeda et. al. (1970) for steel reinforced members.



(a) Primary curve (b) Rules defining unloading and reloading

Figure 5. Moment-displacement hysteretic model of Sharbatdar and Saatcioglu (2007).

The main difference between the hysteretic models for steel and FRP reinforced structures is that the unloading branch for FRP reinforced concrete passes through the origin (no plastic deformation in FRP), as opposed to steel reinforced concrete which can develop permanent plastic deformations upon yielding of steel. The rules for FRP reinforced concrete, defining unloading and reloading stiffnesses within the primary curve are defined as follows;

- Initial loading as well as subsequent loadings beyond the previous displacement follows the primary curve.
- Positive and negative reloading and unloading follows the initial stiffness between positive and negative Δ_1
- Unloading from a point between Δ_1 and Δ_3 aims at a point on the primary curve in the opposite direction, which has a displacement equal to the average of the two previous displacements in the opposite direction, as shown in Fig.5(b). If the previous displacement in the opposite direction has not exceeded Δ_1, unloading would aim at a point with Δ_1 displacement on the opposite primary curve. If previous displacement in the opposite direction has exceeded Δ_1 only once, unloading would aim at a point with a displacement equal to the average of the maximum previous

displacement and Δ_1. Reloading follows the same slope as unloading until the primary curve is reached.

- Beyond Δ_3, the member starts failing either by compression buckling or tension rupturing of FRP reinforcement. If the failure is due to the local buckling of FRP fibres in bars, the element continues to resist loads until Δ_4, at which deformation level the element fails due to the rupture of FRP bars. FRP reinforcement which has failed in compression can not maintain its tensile resistance upon load reversal.

Results and discussion

The two-storey buildings designed with steel and FRP reinforcement were analyzed to conduct a comparative investigation. First the fundamental periods were computed using effective elastic rigidities, including the effects of concrete cracking. The FRP reinforced building was found to have a period of T_{FRP} = 1.0 sec., whereas the steel reinforced building had a period of T_{steel} = 0.48 sec. The corresponding spectral design accelerations were determined from NBCC (2005) to be 64 % lower for the more flexible FRP reinforced building in Ottawa. Similarly the design spectral value for the FRP reinforced building in Vancouver was found to be 62% lower. However, because of the ductility and overstrength related reduction factors, R_d and R_o, the equivalent static design forces for steel reinforced concrete buildings were lower than those for FRP reinforce buildings.

Dynamic inelastic response of each building was established under short-period and long-period hazards. The seismic force demands for the FRP reinforced building in Ottawa were found to be 50% and 28% lower than the companion steel reinforced building under short-period and long-period hazards, respectively. The comparison for buildings designed for Vancouver indicated 38% and 20% lower force demands for the FRP reinforced building under short-period and long-period hazards, respectively. The maximum storey displacements and maximum interstorey drift ratios are plotted in Fig.6. It was found that the maximum interstorey drift ratios were higher for the FRP reinforced buildings, except for the short-period hazard analysis for Ottawa.

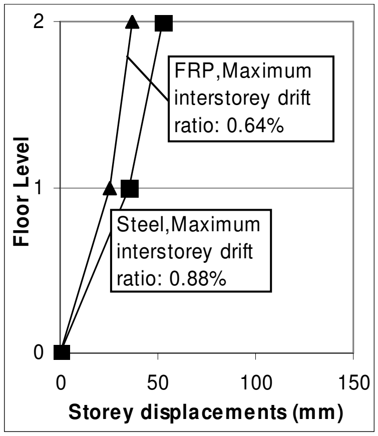
An important aspect of the comparative study was the level of inelasticity experienced in members because of the concerns over ductility of FRP reinforced structures. Fig. 7 shows sample hysteretic relationships obtained from dynamic analysis. It can be observed that in all cases elastic stiffnesses of FRP reinforced members are lower than those of steel reinforced members. This can be explained by the lower modulus of elasticity of FRP reinforcement and the resulting reduction in post cracking stiffnesses. Most beams of steel reinforced structures, especially the first-storey beams experienced inelastic deformations. On the other hand, the beams of FRP reinforced structures remained elastic under the same ground motions while mostly attaining the same deformations as the steel reinforced beams. This can be observed in Fig. 7 (a), (c), (e) and (g).

The columns of steel reinforced structures remained elastic except for some of the first-storey columns of the building in Vancouver, which experienced limited inelasticity. The inelasticity in this building was triggered mostly by the inelastic deformability of confined concrete. The moment-chord rotation diagrams of steel and FRP reinforced columns are compared in Fig.7 (b), (d), (f) and (h).

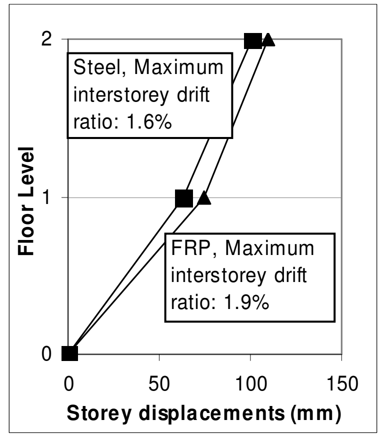
Conclusions

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

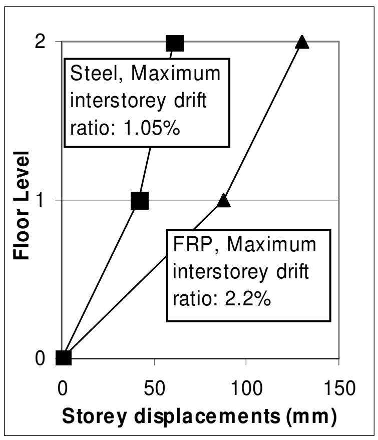
- FRP reinforced concrete structures experience softer behaviour upon cracking due to the lower elastic modulus of FRP reinforcement when compared with conventional steel reinforced concrete structures. Therefore, FRP reinforced buildings may have lower seismic hazards in the mid to long-period range due to the nature of the design spectra. This implies that FRP reinforced concrete structures may attract lower elastic seismic forces than structures with conventional steel reinforcement.



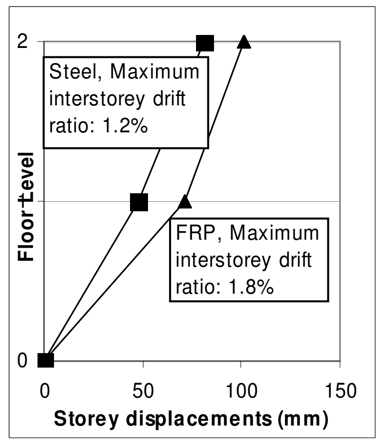
(a) Ottawa short-period hazard



(b) Vancouver short-period hazard



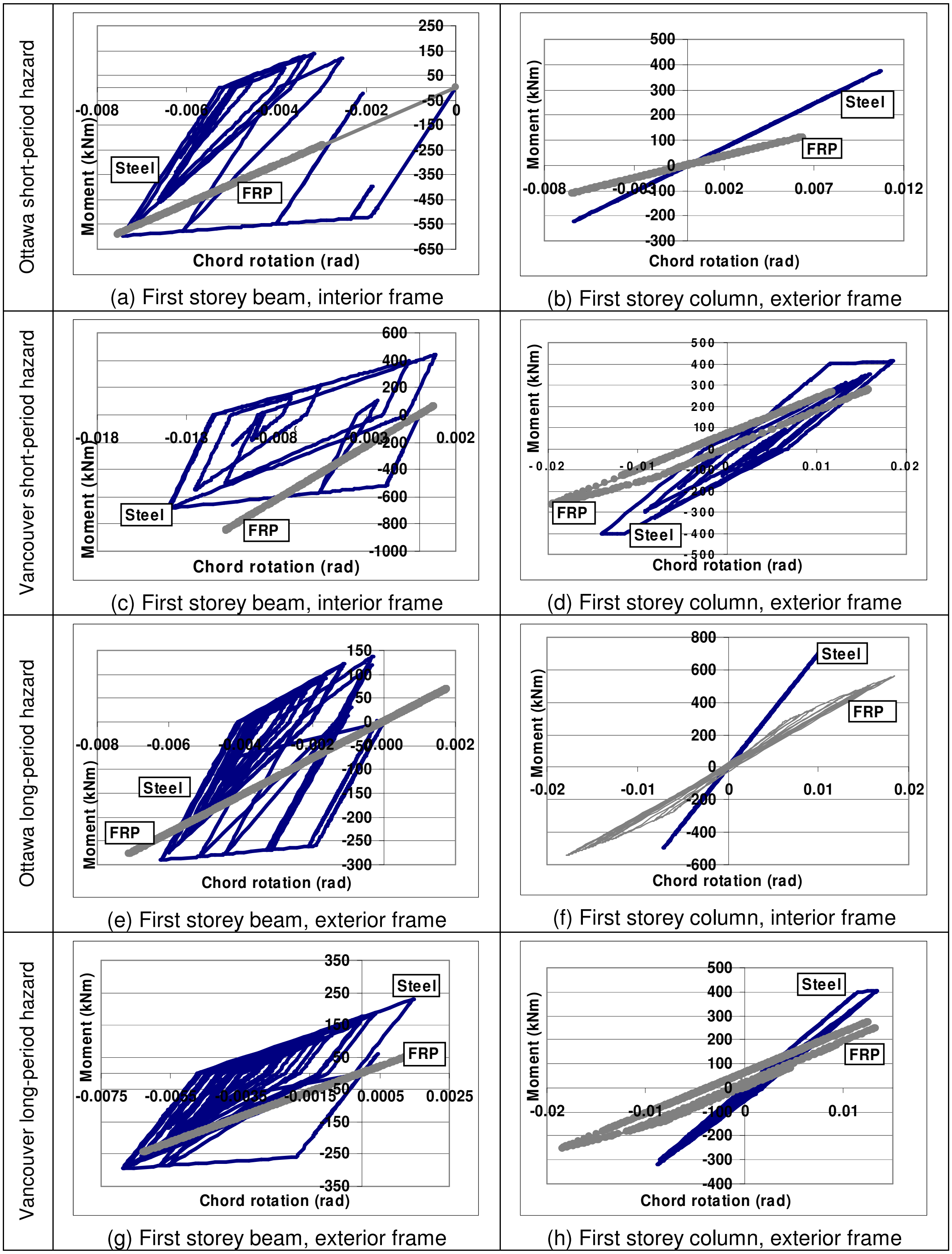
(c) Ottawa long-period hazard



(d) Vancouver long-period hazard

Figure 6. Interstorey drifts for steel and FRP reinforced concrete structures.

- Steel reinforced concrete structures are routinely designed for ductility. However, FRP reinforced structures show brittle behaviour if the failure is triggered by the rupturing of FRP, though limited inelasticity may be present if the concrete is properly confined and the rupturing of FRP in tension is prevented. Elastic seismic force demands for conventional steel reinforced buildings can be reduced because of available ductility and overstrength. However, FRP reinforced concrete structures may have to be designed for elastic response, without any reduction in design force levels, until further research is conducted to justify such reduction. This may result in lower design forces for steel reinforced structures than those for companion FRP reinforced structures.
- Although FRP reinforced buildings are softer, they attract lower seismic forces in the high-period range of design spectra and experience seismic drift rations well within the acceptable range of deformations, while behaving essentially elastic. The maximum drift ratio of FRP reinforced structures considered in the current investigation was 2.2% under NBCC-2005 compatible ground motion records for Vancouver. This value is within the 2.5 % limit specified in NBCC-2005.



(a) First storey beam, interior frame
(b) First storey column, exterior frame
(c) First storey beam, interior frame
(d) First storey column, exterior frame
(e) First storey beam, exterior frame
(f) First storey column, interior frame
(g) First storey beam, exterior frame
(h) First storey column, exterior frame

Figure 7. Moment-chord rotation relationships for selected FRP and steel reinforced members.

- Dynamic response of FRP reinforced buildings under NBCC-2005 compatible ground motions indicate elastic behaviour of beams with limited inelasticty in the columns. This observation indicates that it is possible to design FRP reinforced concrete buildings in seismically active regions. However, more research is needed to devise seismic design guidelines for such buildings.

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