Standard Dynamic Loading Protocols for Seismic Qualification of BRBFs in Eastern and Western Canada

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15 WCEE
LISBOA 2012

SUMMARY:
Standard loading protocols are widely used for seismic performance assessment of structural systems and their components. In this study, a series of standard dynamic loading protocol have been developed through post-processing of nonlinear time-history analyses results. Code-conforming multi-storey buckling-restrained braced frames (BRBFs) were designed, modelled, and subjected to ground motions compatible with different earthquake scenarios in Canada. Histories of the key response parameters including deformation rate were extracted from models and subjected to cycle counting and other statistical post-processing procedures. Finally, standard loading protocols were proposed for intra-plate events in eastern and western regions of Canada, as well as for inter-plate events expected along the west coast of Canada. Earthquake-representative testing rates were obtained based on the statistics of the energy dissipation rates observed in the analyses and incorporated in the loading protocols. The response of the code-conforming BRBFs is also discussed.

Keywords: Buckling-Restrained Braced Frames, Loading history, Ground motion selection and scaling

1. INTRODUCTION

A test loading protocol is a sequence of target deformation amplitudes which represent the effect of earthquake on structural components. It can be applied to real structural components or computer models to verify seismic performance objectives. Loading protocols are widely used in evaluation of innovative structural systems, and seismic rehabilitation of existing structures. Several loading protocols have been developed for a variety of purposes in the past (Krawinkler, 2009). Usually loading protocols are developed based on the simplified response of a specific class of structures or components such as beam-to-column connections for steel moment resisting frames to an expected seismic event. Designed components are supposed to pass the acceptance criteria when they are subjected to relevant loading protocol.

Buckling-restrained braced frame (BRBF) is a lateral system with special bracing elements that do not buckle but yield in compressive loading cycles. Generally this bracing element is composed of a slender ductile core (can be plate or section), a restraining mechanism and/or an un-bounding agent. Bucking of the ductile core is prevented by the restraining mechanism, which results in a stable and symmetrical hysteresis response with greater energy dissipation capacity compared to conventional steel bracing members. Adequate inelastic performance of BRB bracing members heavily depends on fabrication details and tolerances and seismic qualification testing requirements have been introduced for BRBFs in the CSA S16-09 standard for the design of steel structures in Canada (CSA, 2009). CSA S16-09 refers to the loading protocol specified in AISC-341 seismic provisions for BRBFs (AISC, 2005), a testing protocol that was essentially based on the seismic demand imposed on BRBFs in high seismic regions of western U.S. (Sabelli et al., 2003).

Canada has two major different seismic active zones. Western Canada is the most seismically active region of the country and geologically can be classified as an active tectonic region with shallow crustal or subcrustal seismic activities (*intra-plate events*). In addition, off the west coast of Canada there is a long Cascadia subduction zone which is capable of producing deep inter-plate mega-thrust earthquakes (**M**8.0 or greater). On the other hand, eastern Canada is located in a stable continental region with relatively lower seismic activities than the west. However, this region has seen large and damaging intra-plate earthquakes in the past and will inevitably see in the future. The causes of earthquakes in this area are not well understood. Characteristic of the expected earthquakes in these two distinct regions are quite different. In the west, earthquakes are supposed to be larger in magnitude, rich in low frequency content and long in terms of duration. In contrast, shorter-duration and high frequency strong shaking is expected in eastern Canada, and this difference has been shown to affect the structural response (Tremblay and Atkinson, 2001).

Loading protocols for the testing of ductile bracing members have been proposed by (Tremblay and Bouatay, 2002) for Canadian seismic conditions. These protocols have been developed based on the response of braced frame structures designed in accordance with past code editions. Furthermore, new knowledge has been gained on the Canadian seismic hazard in the last decade and there is a need to revisit the loading protocol currently specified for BRBFs in Canadian standard. Preliminary study by (Dehghani and Tremblay, 2012) for western intra-plate earthquakes indicated that the demand would be significantly less than of the AISC 2005 testing protocol. In this study, Multi-storey BRBFs were designed and modelled as prototypes for east and west of Canada. Structural design was carried out using the National Building Code of Canada, NBCC 2010, (NRCC, 2010) and CSA S16-09. Three suites of ground motions compatible with the expected type of events in east and west of the country were compiled and applied to the prototype structures throughout extensive nonlinear time-history analyses. Response parameters were extracted and subjected to further analyses for developing loading protocols. Appropriate deformation rates for each protocol were also estimated and proposed.

2. DESIGN OF PROTOTYPE MODELS

2.1. Description of prototype models

Prototype BRBF structures examined here are typical office buildings located in Victoria (British Columbia) and Montréal (Québec). Victoria represents the highest level of seismic hazard in the west coast of Canada and Montréal has the highest seismic risk among the eastern urban areas of the country. All buildings were supposed to be on soft rock deposit (class C site) and have 36 × 54 m foot print with evenly spaced columns at 9.0 m c/c along the both building axes and have uniform storey height of 4.0 m. The lateral seismic force resisting system in both directions consists of Split-X (or two-storey X) braced frames located on the exterior faces of the building. The Split-X configuration has two specificities: 1) it results in the least beam and column steel tonnage when capacity-based design of BRBF is applied (Dehghani and Tremblay, 2012), and therefore represents a popular arrangement in view of its high effectiveness; and 2) for the same inter-storey drift, braces in Split-X configuration undergo higher strains than braces in an equivalent braced panel with single diagonal bracing members. Split-X configuration was selected for the development of loading protocol because it is likely to be widely used and it represents the worst case scenario in terms of strain demand in the BRB core. Typical effective seismic weight of floors and roof was set to 4.75 and 4.50 kPa, respectively. BRB cores were assumed to be made of ASTM A572, grade 50 (F_y = 345 MPa) steel with R_y = 1.12 (ratio of expected to nominal yield strength). Columns and beams material was supposed to be compatible with ASTM A992 steel (F_y = 345 MPa). First mode period of the western prototype ranges between 0.6 and 1.8 s, and between 0.9 and 2.2 s for the eastern ones. The prototype buildings are designated using the number of storeys followed by W for Victoria (western Canada) or E for Montréal (eastern Canada).

2.2. Modelling assumptions

Two-dimensional models of prototypes were built in SAP2000 structural analysis program (CSI, 2010). BRB cores were modelled using the simplified Bouc-Wen hysteresis law. This model is able to reproduce response of ductile elements with cyclically stable and smooth hysteresis behaviour which is the case for a typical BRB member. The restoring force in this constitutive model is expressed as:

$$F = a\,ku + (1-a)k\,u_y\,z \tag{2.1}$$

where, a = ratio of the post-yielding stiffness to the initial elastic stiffness; k = initial elastic stiffness; u = deformation time-history; u_y = brace yield deformation; and z = hysteresis parameter obtained by solving the following differential equations in time domain:

$$\dot{z}\,u_y = \dot{u}\left[1 - |z|^n\left(\beta_{BW}\,\mathrm{sgn}(\dot{u}z) + \gamma_{BW}\right)\right] \tag{2.2}$$

where, n = yield exponent that controls the sharpness of the transition between initial stiffness and post-yield stiffness; β_{BW} and γ_{BW} = factors to control the shape of hysteresis loops; and sgn = the Signum function. System identification analysis conducted on the actual test results showed that β_{BW} = γ_{BW} = 0.5, n = 2.0, and a = 0.02 are appropriate values. Since isotropic hardening cannot be simulated directly by this model, yield deformation of the system may be modified to approximately adjust the hysteresis law for this effect:

$$u_y = \left[1 + a(\mu_{\max} - 1)\right]R_y F_y A_c / k \tag{2.3}$$

where, $\mu_{\max}$ = maximum expected ductility demand in the brace core; R_yF_y = probable yield strength of the core material; and A_c = BRB core area. In this study, the yield deformation was multiplied by an adjustment factor 1.18, assuming $\mu_{\max}$ = 10. This factor is then readjusted based on the maximum core ductility demand obtained in each analysis case. Inelastic rotational springs were also calibrated and implemented at the ends of the core to simulate flexural response of semi-rigid brace-to-gusset plate connections. Additional modeling assumptions are discussed in (Dehghani and Tremblay, 2012).

3. GROUND MOTION SELECTION AND SCALING

Ground motion selection and scaling has substantial impact on the outcomes of time-history analysis. To overcome the uncertainties associated with selection and scaling of ground motion records, a new approach was developed and implemented in this study. Suites of ground motion records for intra-plate events in Victoria and Montréal were compiled after taking the following steps: 1) dominant events were extracted from hazard deaggregation data; 2) records were selected for each dominant event bin; 3) in each bin, records were ranked based on deviation from central tendencies of the best estimator of intensity, frequency content, and duration; 4) in each bin, the required number of records were selected from the top ranked records; 5) the selected records were then individually linearly scaled over a frequency bandwidth in which most of seismic energy of records is carried on.

3.1. Dominant events

Dominant events were found using available hazard deaggregation data for Canadian cities (Halchuk et al., 2007). Hazard deaggregation technique traces back the seismic hazard (e.g., spectral acceleration) and returns the relative contribution of magnitude/distance pairs in that hazard level. This technique is identified as a probabilistically consistent way for finding dominant earthquake magnitudes and site-to-source distances. Hazard deaggregation data for a given location is usually reported at discrete periods for a specific site soil classification. Deaggregation of 2% in 50 years probability of exceedance of spectral acceleration at period of 1.0 second for class C sites (soft rock) located in Victoria and Montréal are shown in Fig. 3.1. Calculation showed that dominant intra-plate earthquakes for Victoria is a combination of events with 6.25 ≤ M_w ≤ 7.50, and 40 ≤ R ≤ 80 km. Likewise, dominant events for Montréal have a range of 6.0 ≤ m_{bLg} ≤ 7.25, and 20 ≤ R ≤ 120 km. M_w, m_{bLg} and R are moment magnitude, body wave magnitude, and hypocentral distance, respectively.

3.2. Record selection

For every pair of magnitude and distance, consistent records were selected from available ground motion databases. PEER strong ground motion database (PEER, 2011) were used for suite of records representing intra-plate events in Victoria. As it was pointed out earlier, the nature of ground shaking in west and east of North America are quite different. As a matter of fact, databases such as PEER would be suitable for shallow crustal regions such as west of North America. Due to lack of historically recorded earthquakes for eastern Canada, usually combination of historical, hybrid and simulated earthquake records are used. To compile a suite of records for Montréal, two databases were utilized: 1) hybrid ground motions for central and east of the United States (CEUS) (McGuire et al., 2002); and 2) simulated ground motions for eastern North America (Atkinson, 2009). Records for Cascadia inter-plate events were collected from: 1) database of stochastically simulated, and modified subduction records (Atkinson and Macias, 2009); and 2) database of simulated inter-plate events for Cascadia subduction zone (Atkinson, 2009).

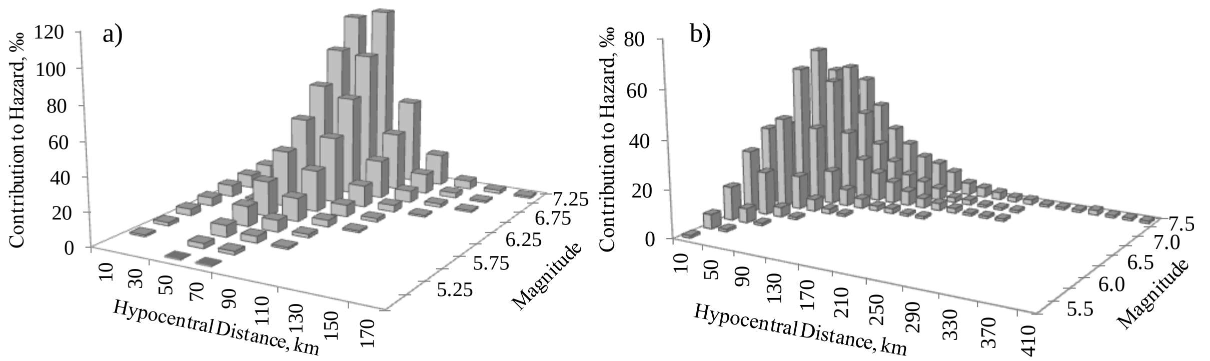

Figure 3.1. Deaggregation of 2% in 50 years seismic hazard at period of 1.0 second for: a) Victoria; b) Montréal

3.3. Selection refinement

When a limited number of seismological metadata such as magnitude, distance and soil class are used to find event-compatible records from a large database such as PEER, several records are obtained, specifically when the magnitude is moderate or low (in order of 6.5 or less). Generally, it is not practical to use all of the event-compatible records in nonlinear time-history analysis because of the computational expenses. Therefore a method is required to refine the initially selected records and reduce them to the required number (usually between 7 and 40). To find the most probable records in terms of damaging seismic signal characteristics, the best estimator parameters representing the ground motion amplitude intensity, frequency content and duration were found using inter-correlation analysis. Arias Intensity, Mean Period and Uniform Duration were found to be typical best estimators for intensity, frequency content, and duration in most cases. Then records were ranked based on average of their normalized deviation from the central tendencies of the best estimators. In this way, records with exceptional ground motion properties are discarded. In this study, 20 intra-plate records for Victoria, 20 intra-plate records for Montréal, and 10 inter-plate records for Victoria were selected throughout the proposed refinement procedure.

3.4. Scaling of the selected records

Finding a proper method for scaling of ground motion records still represents a challenge in earthquake engineering. Recent studies have shown that conventional scaling procedures are conservative and lead to overestimated and widely scattered structural responses (NIST, 2012). In the framework of this study, a method for linear amplitude scaling in time domain referred to herein as Least Moving Average was developed and implemented. This method is not dependent on the fundamental frequency of the structure. Simply this method looks for a narrow frequency bandwidth in which the shape of the as-recorded acceleration spectrum is very similar to the target spectral shape. In this way, the ground motion is scaled at a frequency range in which seismic energy is concentrated. To find this frequency or period range, the ratio of the target spectral ordinate (S_a^{Target}) to the 5% damped spectral acceleration of the record (S_a^{GM}) is calculated first. The average (S_a^{Target} / S_a^{GM}) ratios over a period range from $0.5T_i$ to $1.5T_i$ are then computed at every period T_i in the range of 0.2 to 4.0 s. The minimum average ratio is finally used as the scaling factor. Backward and forward averaging is meant to smooth the jagged shape of ground motion spectrum. Past studies have shown that using this algorithm, high frequency records are generally scaled in the high frequency band of the spectrum and low frequency motions are scaled in the low frequency region of the target spectrum. Layout of the proposed scaling method is shown in Fig. 3.2 for strong ground shaking recorded at UCSC 13 BRAN station in the 1989 Loma Prieta earthquake (**M**6.9).

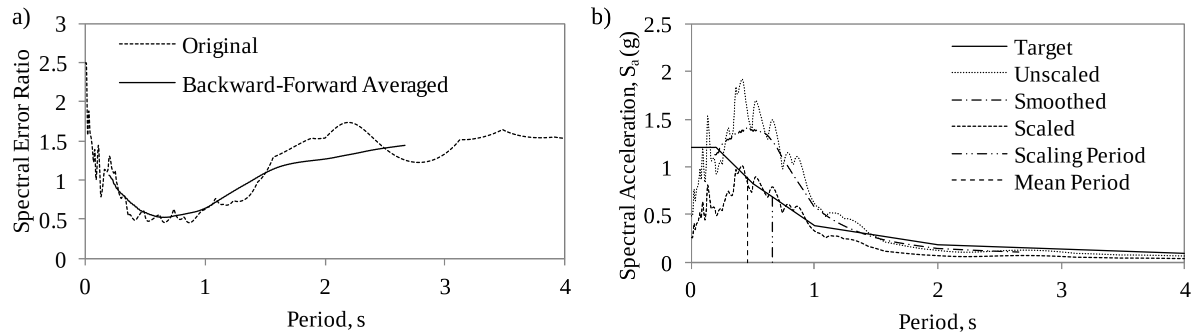

Figure 3.2. Layout of the Least Moving Average ground motion scaling method

Generally, this scaling method gives lower scaling factors than conventional methods such as scaling at the fundamental period of the structure. Mean and 84th percentile of scaled acceleration spectra of the selected records for intra-plate events are compared with the NBCC 2010 2% in 50 years uniform hazard design spectrum for Victoria and Montréal in Fig. 3.3a and b. Since inter-plate seismicity was not included in the probabilistic NBCC 2010 uniform hazard spectrum, records from these events were scaled to the deterministic Cascadia hazard (see Fig. 3.3c) as reported in (Halchuk and Adams, 2003).

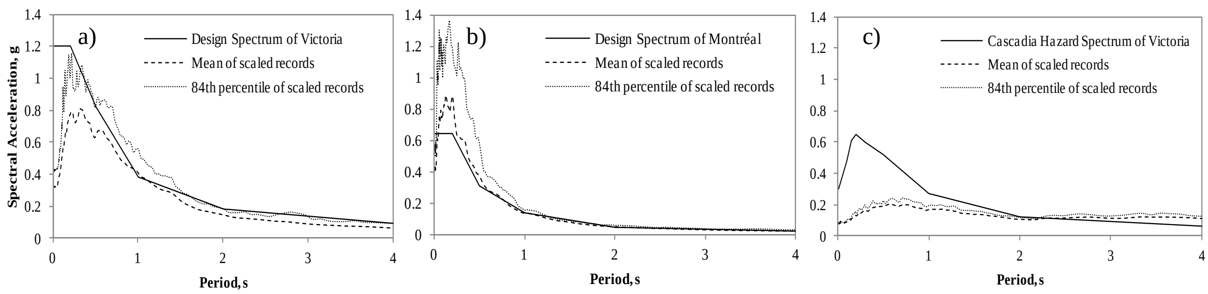

Figure 3.3. Spectra of scaled records: a) Victoria intra-plate; b) Montréal Intra-plate; c) Victoria Inter-plate

4. ANALYSIS SETTING AND RESULTS

4.1. Analysis assumptions

Nonlinear time-history analyses were conducted using Newmark transient integrator (Gamma = 1/2, Beta = 1/4). Rayleigh damping coefficients were tuned based on 3% critical damping ratio between the fundamental vibration mode and the mode of 98% accumulated modal mass. A special technique is used to avoid generation of large artificial stiffness-proportional damping forces in the columns. Since accidental torsion effects were included in the design of the prototype structures, records were further scaled up by factor of 1.06 to achieve consistency between the demand and the design assumptions. In total, 200 time-history analysis cases were carried out and 4800 brace axial deformation histories were generated in the course of the study. Time histories of key global and local response parameters such as inter-storey drifts, brace axial deformations, storey relative velocities and absolute accelerations, and internal actions in columns were also extracted from the computer models. In addition, all prototype buildings were subjected to nonlinear static analysis (pushover) with inverted triangular force pattern. Roof target displacements for a given prototype were calculated based on:

$$u_{roof}^{\text{Target}} = \left[1 + a\left(\mu_{\max} - 1\right)\right] R_y \sum_{i}^{n} \varphi_{i,roof} \Gamma_i S_d(T_i) \tag{4.1}$$

where, i = mode of vibration; n = mode of 98% accumulated mass; $\varphi_{i,roof}$ = mode shape factor at roof level; Γ_i = modal participation factor; T_i = period of i^{th} mode; and $S_d\,(T_i)$ = 5% damped elastic design spectral displacement at the i^{th} mode.

4.2. Global performance

Global behaviour of the prototype structures were evaluated using performance indicators such as peak amplitude of inter-storey drift [θ_{max}], maximum accumulated drift [$(\Sigma\theta)_{max}$], maximum residual drift [$(\theta_{res.})_{max}$], maximum storey absolute acceleration [a_{max}], highest instantaneous inter-storey relative velocity [v_{max}], highest inter-storey inelastic deformation rate [$(\Delta u/\Delta t)_{max}$], maximum base shear ratio [$(V/V_{design})_{max}$], and minimum safety factor on overturning moment. Table 4.1 summarizes the statistics of the global performance indicators for the prototypes subjected to different ground motion suites.

Table 4.1. Statistics of the global performance parameters in all analysis cases

Global performance parameters	West Intra-plate Events			East Intra-plate Events			West Inter-plate Events		
	Max.	Mean	COV	Max.	Mean	COV	Max.	Mean	COV
a_{max} (g)	0.7	0.4	26%	0.7	0.2	41%	0.3	0.2	22%
θ_{max} (%)	2.4%	0.9%	40%	1.1%	0.4%	35%	1.2%	0.5%	18%
v_{max} (cm/s)	42	22	33%	39	17	39%	18	9	30%
$\Delta\theta_{max}$ (%)	2.4%	1.2%	29%	1.4%	0.6%	36%	1.1%	0.7%	13%
$(\Sigma\theta)_{max}$ (rad.)	0.911	0.299	39%	0.517	0.091	90%	1.381	0.675	30%
$(\Delta u/\Delta t)_{max}$ (cm/s)	30	14	34%	26	12	41%	12	6	32%
$(\theta_{res.})_{max}$ (%)	2.3%	0.3%	94%	0.7%	0.1%	97%	1.0%	0.1%	84%
$(V/V_{design})_{max}$	1.6	1.4	6%	1.5	1.1	15%	1.4	1.2	7%
$(SF_{Movr})_{min}$	3.5	4.2	12%	9.2	13.6	20%	4.3	5.0	7%

4.2.1. Victoria under intra-plate events

In the extreme case, prototype 9W experienced the largest inter-storey drift ratio at its first storey (2.4%), which is lower than the allowable design limit, and the previously reported values in the literature (Sabelli et al., 2003). These lower drift values can be attributed to the ground motion selection and scaling method adopted in this study, different lateral force modification factor [R_oR_d = 4.8 (Canada); R = 7.0 (US)], and the type of hazard (e.g., presence of near-field hazard in California). Generally, damage is more concentrated at the first and the last storeys. Maximum storey absolute accelerations were bounded to 0.69g (in prototype 5W). Inter-storey inelastic deformation rate of 30 cm/s was experienced at the top storey of the prototype 5W. The maximum base shear ratio of 1.6 observed in the prototype 3W and the safety factors against overturning were generally high. In Fig. 4.1a, the design inter-storey drift profile for prototype 9W is compared to the results of the pushover analysis and the envelope of the peak inter-storey drifts in all nonlinear time-history analysis cases. The pushover analysis predicts excessive concentration of deformation demand in the lower storeys. This demand is quite different from the more uniform demand obtained in the time-history analyses. Seismic demands observed in this study are higher than the ones found in the preliminary research of the authors (Dehghani and Tremblay, 2012). This implies the possible impacts of the ground motion selection on the demand.

4.2.2. Montréal under intra-plate events

The extent of damage in eastern Canada prototypes was much lower than in the western counterparts. In fact, maximum inter-storey drift was limited to 1.1% (the last storey of 3E) which is well below the design limit (2.5%). As discussed earlier, eastern intra-plate events are supposed to be rich in high frequency motions and are less likely to drive large inelastic deformations in flexible structures. In addition, NBCC 2010 requires that the base shear for the eastern prototypes be amplified for higher mode effects. This extra amplification results in higher system overstrength when compared with the western prototypes. In general, hybrid records imposed more demand than the simulated ones.

4.2.3. Victoria under inter-plate events

Seismic demands under inter-plate events were always lower than the intra-plate counterparts except for the accumulative drift. The main reason for this trend is the lower design accelerations for the inter-plate events. In fact, Cascadia deterministic spectral ordinates of Victoria are always lower than the UHS design spectrum, specifically in the short period region of the spectrum. As a result, low-rise prototypes were only slightly damaged. The maximum inter-storey drift was observed at the first storey of prototype 9W (1.2%). The maximum accumulated damage is approximately equivalent to 80 fully reversed yield cycles observed at the top storey of prototype 9W.

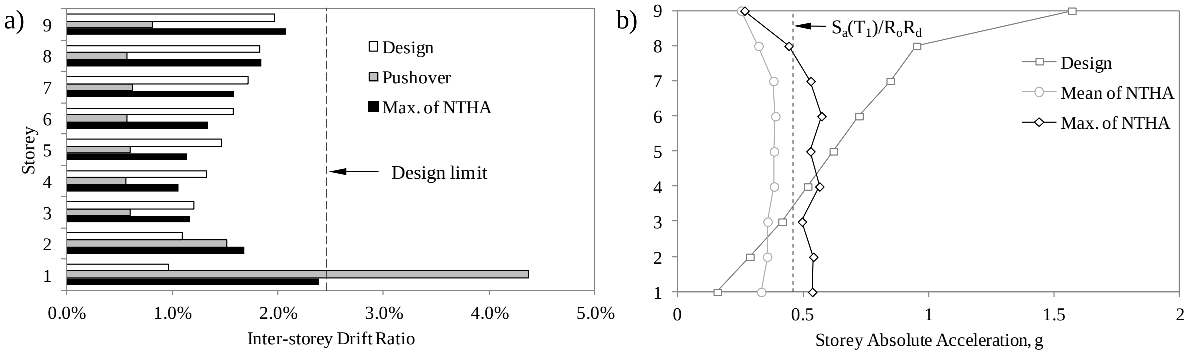

Figure 4.1. Comparison between predicted and computed demands by for prototype 9W

4.3. Local performance

Local performance was evaluated using deformation history of the brace cores. Since stiffness of the projected areas of the core were included in the brace element, the following equation was used to recover net core ductility demand:

$$\mu = \frac{\Delta_c - \Delta_p}{\Delta_{y,core}} = \frac{\Delta_c}{\gamma \varepsilon_y L_{br}} - \eta \frac{(1-\gamma)}{\gamma} \frac{P}{P_y} \tag{4.2}$$

where, Δ_c = computed brace element deformation; Δ_p = elastic deformation of the projected area; γ = ratio of the yielding length to the brace total length; ε_y = probable yield strain of the brace core modified for isotropic strain hardening effect; L_{br} = brace length in the model; η = ratio of the core cross-section area to the cross sectional area of the projected segment; P = brace axial force history; and P_y = probable yield strength of the brace core. Ductility histories were then converted to peak-and-valley data from which the following demand indices could be extracted: 1) peak ductility ($|\mu|_{max}$); 2) maximum ductility excursion ($|\Delta\mu|_{max}$); 3) maximum ductility range ($|\mu_{max} - \mu_{min}|$); 4) cumulative ductility ($\Sigma|\Delta\mu|$); 5) maximum ductility rate ($|\Delta\mu/\Delta t|_{max}$); and 6) normalized hysteresis energy ($\Sigma(\mu{\times}P/P_y)$). These demand indices were used to find the most damaged brace cores. Statistics of the maximum demand for all prototypes in all analysis cases are shown in Table 4.2. These values were considered when finalizing the loading protocols.

Table 4.2. Statistics of the seismic demands on the buckling-restrained brace cores

Demand Indices	West Intra-plate Events			East Intra-plate Events			West Inter-plate Events		
	Max.	Mean	COV	Max.	Mean	COV	Max.	Mean	COV
$\|\mu\|_{max}$	10.5	3.7	43%	6.5	2.3	45%	6.0	2.9	28%
$\|\Delta\mu\|_{max}$	9.6	3.7	35%	5.8	2.1	41%	3.6	2.1	14%
$\|\mu_{max} - \mu_{min}\|$	11.6	4.7	34%	6.2	2.7	34%	6.7	3.3	18%
$\Sigma\|\Delta\mu\|$	270	74	44%	161	23	104%	239	130	30%
$\|\Delta\mu/\Delta t\|_{max}$	28	12	38%	22	10	43%	9	4	36%
$\Sigma(\mu{\times}P/P_y)$	51	9	69%	28	2	116%	13	5	41%

5. LOADING PROTOCOLS

For each suite of records, the ductility histories for the most damaged brace cores were subjected to rain-flow cycle counting. This method converts an irregular signal into clustered bins of amplitudes-and-means, and the number of cycles for each bin. Results of the cycle counting were then simplified by neglecting the offset residual deformations, and the deformation sequence effect. The values of the demand indices were used to qualify and adjust the amplitudes and the number of cycles of the obtained loading protocols. Cycle counting showed that number of inelastic excursions exponentially decreases as their amplitude becomes larger. The intensity and extent of the seismic demand on structures are related to a number of factors including frequency content of the seismic event, site soil behaviour, configuration of structure and its components, dynamic properties of buildings, etc. Ideally, any standard loading protocol should address all these issues. However, it is not possible to include all these factors and their variability in one single loading protocol. This requires the loading protocols to be generally conservative. To be on the conservative side, the number of low amplitude cycles was reduced and the cumulative ductility associated with these removed cycles was distributed between higher amplitude cycles. The final loading protocols are shown in Fig. 6.2.

6. DEFORMATION RATES

Past experimental studies have shown that fast deformation rates (such as those expected under intense earthquakes) can increase the yield strength of structural steel (Bruneau et al., 1998). This increased yield strength may amplify the force demand on the force-controlled elements such as connections, columns and beams. High deformation rates may also cause damage concentration, and give rise to the brittle modes of fracture (Fell, 2008). Popular standard loading protocols do not suggest testing rates and, as a result, structural assemblies are allowed to be qualified under quasi-static rates. This may not reveal the actual seismic performance under the expected earthquake deformation rates. To include appropriate deformation rates in the developed protocols, deformation histories of the most damaged brace cores were subjected to further analyses. Two independent approaches were adopted in this study: 1) statistical analysis of deformation rates in the moderate and large inelastic excursions; and 2) average of effective duration of the plastic work.

6.1. Statistical analysis of deformation rates

At the beginning, the small amplitude excursions (e.g., $|\Delta\mu/2| \leq 0.50$) were discarded from the given ductility signal. Ductility rate of the remaining signal was then obtained and stored. Further analysis gave the statistical distribution of the ductility rates given that the ductility amplitude is known. Since deformation rate is related to the dynamic characteristics of the studied prototypes and the frequency content of the ground motions, wide variability in the deformation rate, particularly in the lower amplitudes, was observed. As a conservative decision, the 84th percentile value of the rates for a given ductility amplitude was considered for the loading protocols. Distribution of these rates for different types of event is shown in Fig. 6.1. The method gives a range of 10 to 20 ductility-per-second for the west intra-plate events. This range shifts to slightly faster rates for the east intra-plate events and much slower rates (with more variability) for the west inter-plate events. In fact, this approach suggests application of multiple loading frequencies for given amplitude, specifically the lower ones.

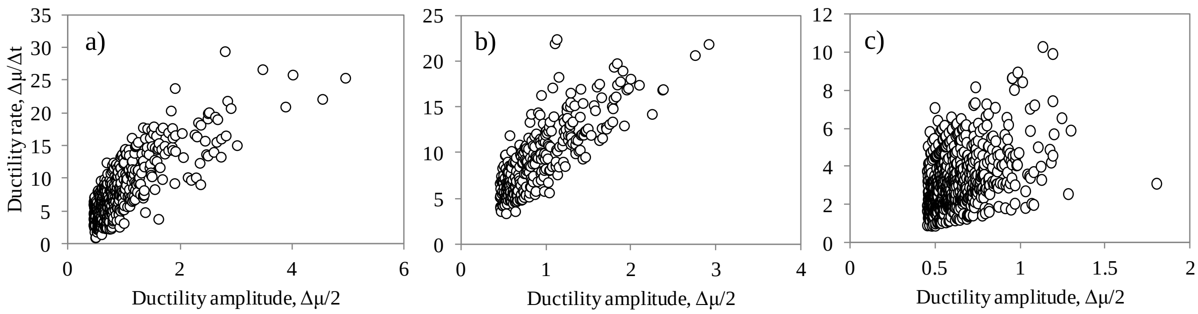

Figure 6.1. Distribution of ductility rate for: a) west intra-plate; b) east intra-plate; and c) west inter-plate events

6.2. Effective duration of plastic work

The average time required to accumulate 5 to 95 percent of the total plastic work in the most damaged brace cores was taken as the total testing duration. This duration can be considered as the required time for the large inelastic deformation cycles to be completed. Given this duration, testing deformation rate can be established assuming the same average ductility rate for all amplitudes of the loading protocol. As a result, frequency of loading decreases as the deformation amplitude becomes higher. This is consistent with the observations in the first approach while being more practical to execute. Based on this concept, constant deformation rate for each type of events was computed and applied to the corresponding loading protocol (Fig. 6.2). This rate is equal 11.5, 16.5, and 1.3 ductility-per-second for west intra-plate, east intra-plate, and west inter-plate events, respectively.

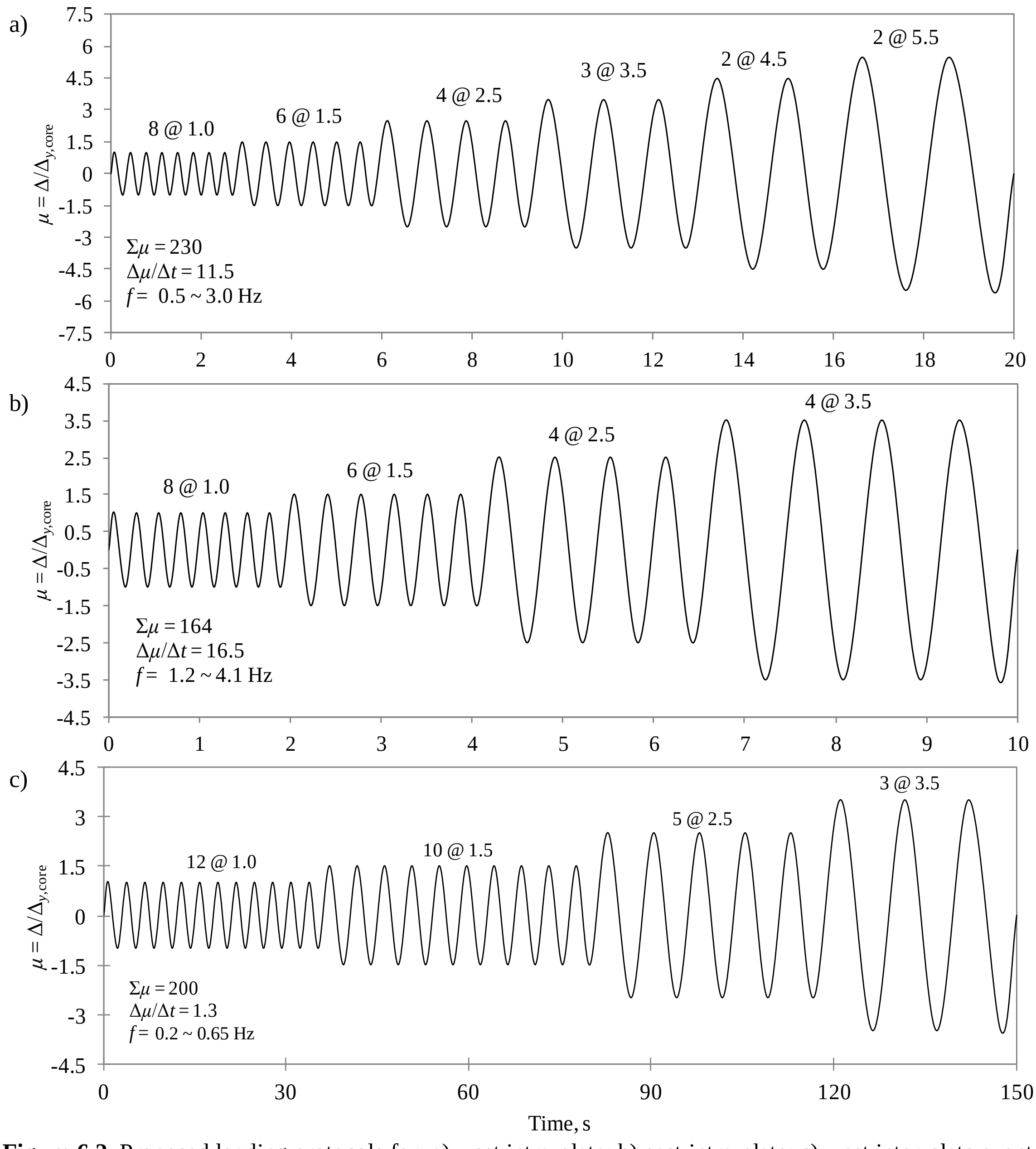

Figure 6.2. Proposed loading protocols for: a) west intra-plate; b) east intra-plate; c) west inter-plate events.

7. CONCLUSIONS

Series of standard dynamic loading protocol for testing buckling-restrained brace elements for the eastern and western regions of Canada were developed. Different possible hazard scenarios and the effects of local seismicity were addressed in the process of ground motion selection and scaling. Time-history analyses showed that seismic performance of code-conforming BRBFs is satisfactory. Western intra-plate events have shown to be more damaging than the other two types of events. Analysis results also showed that even in the worst case, inter-storey drift ratios remain lower than previously reported values for case studies designed and analyzed for California. Seismic demand has shown to be more concentrated at the first and the last storeys of the studied structures. From a statistical assessment of key response parameters for the prototype buildings studied, testing protocols with different displacement (ductility) amplitudes, number of cycles, loading rates and durations have been proposed for each type of Canadian seismic hazard.

ACKNOWLEDGMENT
The financial support provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) for the Canadian Seismic Research Network (CSRN) is acknowledged. Scholarship awarded to the first author by Structural Steel Education Foundation (SSEF) is also acknowledged.

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