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IMPROVED EARTHQUAKE RESILIENCE WITH PERFORMANCE-BASED SEISMIC DESIGN IN CANADA

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Abstract: The focus of current Canadian traditional code-based seismic design is to primarily ensure life safety of building occupants. This objective is currently achieved by satisfying prescriptive criteria expressed in terms of drift and strength requirements as prescribed by the National Building Code of Canada or the Building Code in effect under the applicable jurisdiction. However, earthquakes in the last decade such as Chile (2010) and New Zealand (2011) have clearly demonstrated that these natural disasters can have devastating social and economic impacts. Many of the large urban centres in Canada are located within zones of moderate to high seismicity and are susceptible to the effects of major seismic events. Whereas conventional code-based design focuses on meeting prescriptive requirements, Performance-Based Seismic Design (PBSD), a relatively new and emerging seismic design methodology, provides various stakeholders with a better appreciation of the expected performance of a building during a seismic event, often expressed for multiple levels of ground shaking. This paper provides an overview of the state of practice in performance-based seismic design of buildings in North America, with an emphasis on reinforced concrete shear wall buildings. Topics explored include current Canadian code-based seismic design procedures, existing North American performance-based seismic design guidelines, targeted building performance levels, hazard levels, performance objectives and acceptance criteria. This methodology is then briefly examined for adaptation to a Canadian design context. Needs for future research are also identified.

1 INTRODUCTION

Whereas conventional code-based design relies on prescriptive requirements which focus primarily on life safety of the building occupants, Performance-Based Seismic Design (PBSD), a relatively new and emerging seismic design methodology, considers seismic design at different intensities of ground shaking with various performance criteria. This approach provides stakeholders with a better appreciation of the expected performance of a building during a seismic event, often expressed for multiple levels of ground shaking. This paper outlines the state-of-practice in performance-based seismic design of buildings, with an emphasis on reinforced concrete shear wall buildings. Topics explored include a brief review of current Canadian code-based seismic design procedures, an overview of performance-based seismic design methodology and targeted building performance level as well as a short review of the latest available North American guidelines on PBSD. Needs for future research are also identified.

2 OVERVIEW OF PERFORMANCE-BASED SEISMIC DESIGN

Over the last two decades, several guidelines have emerged in the United States with a PBSD framework. Although PBSD was originally developed for the assessment and evaluation of existing buildings (ATC 1996, FEMA 1997, FEMA 2000 and ASCE 2017b), there has now been a shift in the last decade towards incorporating this framework for the next generation analysis and earthquake design of new buildings (PEER 2017 and LATBSDC 2017).

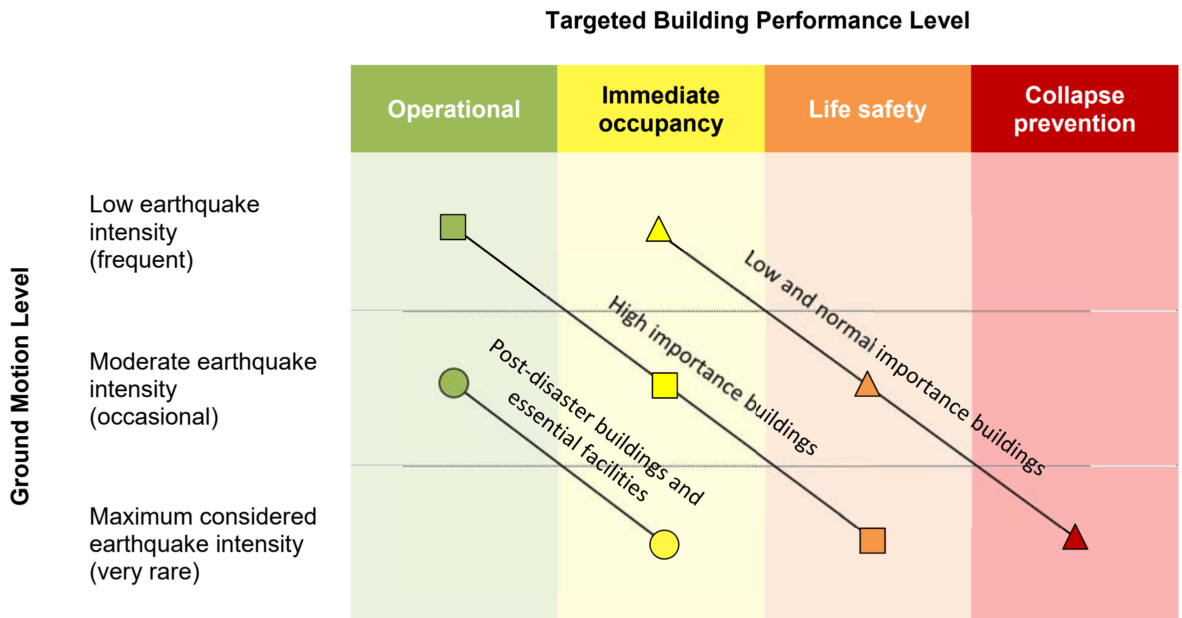

Figure 1: Performance-Based Seismic Design Concept
Adapted from FEMA 450 (FEMA 2004)

As illustrated in Figure 1, a performance-based seismic design framework consists of evaluating the seismic performance of a building for multiple levels of ground shaking. Acceptance criteria are set for each targeted building performance for each ground motion level. Also, different targeted building performance levels apply at a given intensity of shaking as a function of a design category (shown here as a function of the building importance category). This emerging methodology aims to provide various stakeholders with a better appreciation of the expected performance of a building during a seismic event for a given design category. In addition, this framework also enables building owners to choose a design approach which aims at attaining a better performance than the minimum prescribed by Code. Given the complexity of this procedure over traditional code-based methods, it requires extensive knowledge in the characterization of ground motions (tectonic environment, earthquake magnitude, type of faulting/sources, ground attenuation relationships and local site conditions), the nonlinear behaviour of structural materials under reverse cyclic loading (post-yield and post-capping), dynamic modeling and the response evaluation of the structural components.

2.1 Targeted Performance Levels

In the PBSD approach developed for the seismic assessment and rehabilitation of existing buildings (ATC 40, FEMA 273, FEMA 356 and ASCE 41), targeted building performance levels and corresponding objectives were established with the pairing of a structural performance level with a non-structural performance level. Commonly recognized structural performance levels in the above noted publications include immediate occupancy, damage control, life safety, limited safety, structural stability and not considered. The objectives covered under these levels relate to the performance of the structural components and are expressed qualitatively. For concrete shear wall SFRSs, these objectives are mostly described in terms of expected concrete cracking, spalling and/or crushing as well as damage to boundary elements. On the other hand, non-structural performance levels generally consist of operational, position retention, life safety and not considered. These levels cover qualitative objectives related to non-structural components such as partitions, ceilings, mechanical and electrical equipment and other architectural ornamentations.

While several combinations of structural and non-structural levels are identified, it is inferred (ASCE 2017b) that only a select few pairings are considered to provide a balanced design approach. The main targeted building performance levels are Operational, Immediate Occupancy, Life Safety and Collapse Prevention with objectives expressed qualitatively in terms of drifts, strength / stiffness retention and function of the gravity load-carrying elements post-earthquake. Buildings designed to the highest building performance level (Operational Level) are expected to retain most of their pre-earthquake strength and stiffness and have no permanent drifts. Although very light damage may occur, it is noted (ASCE 2017b) that this should not impede on the operation and functionality of the building. At the other end of the spectrum, buildings designed to the lowest performance level (Collapse Prevention) are expected to sustain significant and extensive damage during high intensity ground shaking. Despite significant loss in strength / stiffness of the SFRS elements and expected large permanent drifts, the load-bearing structural elements should still be able to support the gravity loads. It is noted that buildings designed under this performance level are essentially expected to be near collapse and it is recommended to prohibit occupancy in these buildings post-earthquake (ASCE 2017b). As illustrated in Figure 2, the higher the selected performance, the lower the anticipated loss is.

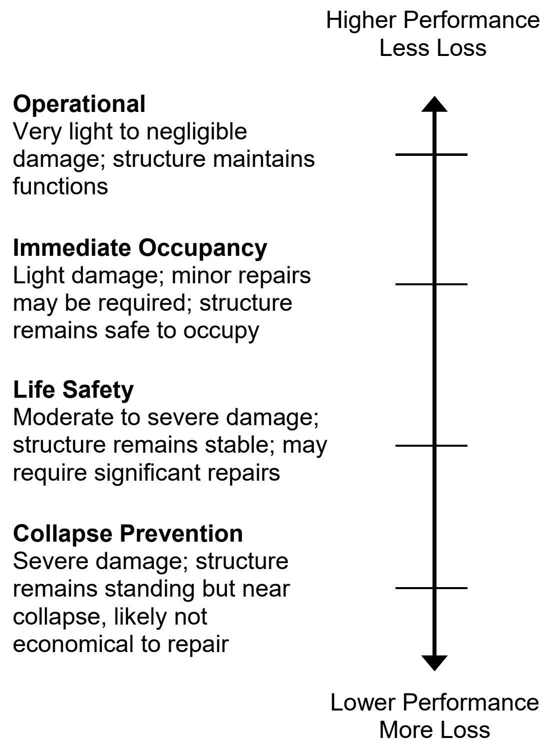

Figure 2: Expected Post-Earthquake Damage State for Various Building Performance Levels (Adapted from Table C2-1 of ASCE 2017b)

In essence, performance levels identify what to expect from a structure as it responds to various intensities of ground shaking. Although performance objectives have remained relatively unchanged over the years, it is worth noting that the initial edition ASCE 41-06 and its predecessor FEMA 273 provided insights in terms of expected drifts, which are summarized in Table 1 for concrete frames and shear wall SFRSs. These target indicators were removed in subsequent editions. A generic force-deformation response is also illustrated schematically in Figure 3.

Table 1: Structural performance level drifts

Seismic Force Resisting System (SFRS)	**Type of drift**	**Expected drifts**		
		Immediate Occupancy	**Life Safety**	**Collapse Prevention**
Concrete frames	Transient Permanent	1% negligible	2% 1%	4% 4%
Shear walls	Transient Permanent	0.5% negligible	1.5% 0.5%	2% 2%

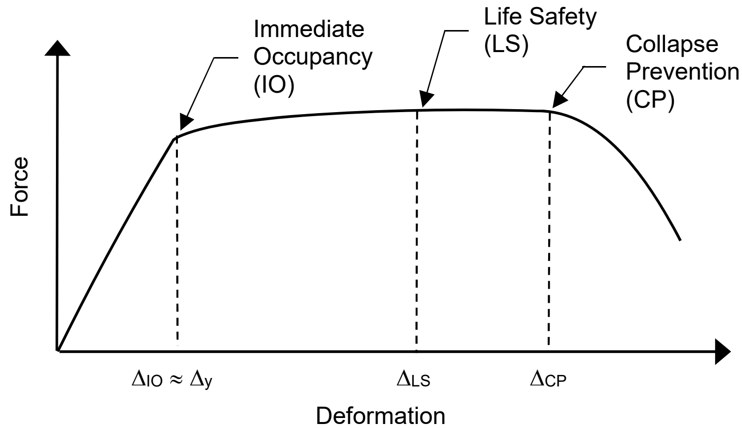

Figure 3: Schematic of structural performance levels and associated deformations

It is generally considered the norm currently in Canada to design buildings for a targeted building performance level of “Life Safety” at the maximum considered level of ground shaking; however, buildings of greater importance such as hospitals are required to remain functional post-earthquake. As such, these buildings are typically designed for a higher performance level to remain functional at the same level of ground shaking.

2.2 Hazard Levels

Current traditional code-based seismic provisions only focus on one hazard level. In Canada, the NBCC requires consideration to a hazard level corresponding to a probability of exceedance of 2% in 50 years. In the US, the requirements of Chapter 16 of ASCE 7-16 on nonlinear response history analysis (NLRHA) specify a design earthquake level which corresponds to the Maximum Considered Earthquake (MCE_R) where MCE_R is based on a risk adjusted uniform hazard with probability of exceedance of 2% in 50 years and deterministic ground motions. Other types of seismic analyses in ASCE 7 consider a design level earthquake which is taken as 2/3 MCE_R. For the analysis and design of new buildings, recent PBSD guidelines for tall buildings (PEER 2017 and LATBSDC 2017) recommend the use of two hazard levels corresponding to probabilities of exceedance of 50% in 30 years and an ASCE 7-16 MCE_R level, for service level and MCE_R evaluations, respectively. On the other hand, various hazard levels are considered in ASCE 41-17: ASCE 7-16’s MCE_R, 2/3rds MCE_R, 5% in 50 years, and 20% in 50 years.

3 CURRENT CODE DESIGN PROCEDURE

Building regulations provide minimum requirements that need to be met for the construction of new buildings. In Canada, building codes mandated by municipalities and provinces are typically based on the adoption of Canada’s National Model Construction Code, the National Building Code of Canada (NBCC), with some modifications in certain jurisdictions. Canada adopted its first building code in 1941 with an empirical formula for seismic design covered under the Code’s appendix. The very simplistic static force formulation consisted of computing the product of the building weight and a constant expressed as a function of the soil bearing capacity, with the resulting horizontal force applied at the center of gravity of the structure.

Although different, the current 2015 NBCC (NRCC 2015) continues to provide code users with a traditional code-based static approach for earthquake design. This conventional approach, known as the Equivalent Static Force Procedure (ESFP), consists of computing the elastic earthquake demand based on uniform hazard values having a probability of exceedance of 2% in 50 years and adjusting it to account for the anticipated ductility (R_d) and overstrength (R_o) of the chosen seismic force resisting system (SFRS). Where permitted, buildings designed under the ESFP are required to resist a lateral earthquake force, V, distributed over the height of the building as a function of the ratio of the product of storey weight and height relative to the summation of this product over the entire height of the building. The lateral force is also adjusted for soil type, higher mode effects and to account for the type of building using an importance factor.

Despite that the ESFP is permitted in certain cases, the default method of analysis in the 2015 NBCC is the Dynamic Analysis Procedure. Under this procedure, buildings may be designed using linear dynamic analysis (either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method) or Nonlinear Dynamic Analysis. Where Linear Dynamic Analysis is used, the analysis elastic base shear, V_e, is to be modified to account for the importance category of the building and the ductility of the SFRS by multiplying the resulting analysis shear by the importance factor, I_E, and dividing it by R_dR_o to obtain the design base shear, V_d. For regular and irregular structures, the minimum base shear determined from the Linear Dynamic Analysis, may not be taken less than 80% and 100%, respectively, of the base shear, V, determined from the ESFP described above. The elastic storey shears, storey forces, member forces and deflections obtained from a Linear Dynamic Analysis are also to be multiplied by the ratio V_d/V_e to obtained the design values.

Very little information is provided in the current NBCC with respect to linear and nonlinear time history analyses. For linear analysis, the code provisions specify that ground motion histories used must be compatible with the code-specified response spectrum; whereas, for nonlinear analysis, the code simply indicates that a special study is required. Despite the limited information in the actual body of the NBCC, a new Annex on the "Selection and Scaling of Ground Motions" has been introduced in Commentary J of the User's Guide – 2015 NBC Structural Commentaries (NRCC 2017). This Annex provides guidance on how to develop the target response spectrum, determine the applicable period range and the number of ground motions required as well as methods for scaling the ground motions to be used for time history analyses to ensure adequate and target response spectrum compatible motions are used. Guidance is also provided in the Commentary on determining force and deformation demands. A summary of selected requirements of the Annex is shown in Table 2. These are also discussed in greater detail in the Section 4.

Except for restrictions, design limitations and other design requirements, two acceptance criteria are noted in the NBCC: a global drift criterion and a general strength criterion. The global acceptance criterion specifies an interstorey drift limit of 1% for post-disaster buildings, 2% for high importance buildings and 2.5% for other types of buildings. The strength criterion simply specifies that the factored resistance of structural components must be greater than the factored demand. Whereas the NBCC specifies requirement for design loads, the structural resistance and detailing requirements for buildings typically fall under the material CSA design standards. The latest seismic design and detailing requirements for concrete buildings are specified under Clause 21 of the CSA A23.3-14 design standard, Design of Concrete Structures (CSA 2014). These requirements are consistent with a capacity design approach where pre-determined elements of the SFRS are designed and detailed to allow energy dissipation to occur with the remainder of the structural elements designed for amplified forces to ensure that the targeted energy dissipating mechanism is obtained. For example, flexural shear walls are designed with an intended flexural hinge mechanism typically located at the base. These walls are then required to be capacity protected in shear by amplifying the shear design force in consideration to the actual flexural capacity of the shear wall.

4 REVIEW OF NORTH AMERICAN NONLINEAR RESPONSE HISTORY ANALYSIS PROVISIONS AND PERFORMANCE-BASED SEISMIC DESIGN GUIDELINES

Several documents exist on the topic of performance-based seismic design and nonlinear response history analysis (NLRHA). This section of the paper specifically focuses and provides a brief overview of the following documents:

- "Chapter 16 – Nonlinear Response History Analysis" – ASCE 7-16,
- "Seismic Evaluation and Retrofit of Existing Buildings" – ASCE 41-17,
- "TBI Guidelines on the Performance-Based Seismic Design of Tall Buildings" – PEER (2017), and
- "National Building Code of Canada 2015 Nonlinear Time History Analysis" – NRCC (2015) and "User's Guide – NBC 2015 Structural Commentaries, Commentary J" – NRCC (2017).

The overarching goals, ground motion characterization procedures, modeling parameters and acceptance criteria are discussed below, summary of which is included in Table 2. It is important to note that the table does not cover all requirements and/or exceptions. For a comprehensive list of requirements, the reader is referred to the publications.

4.1 Performance Objectives

Performance objectives vary amongst the above noted publications. The intent of the PEER (2017) guidelines, as noted in this document, is to provide designers with a performance-based seismic design approach for the design of tall buildings as an alternative to the prescriptive code-based procedures of ASCE 7-16. It is noted that buildings designed in accordance with these guidelines are expected to remain essentially elastic under the service level earthquake (SLE) and to respond to the MCE_R intensity shaking without the loss of the gravity load system (i.e. collapse prevention). The scope of these guidelines is however limited to tall buildings, which is defined as those having a lateral fundamental period greater than 1.0s, a high mass participation and lateral response in higher modes of vibration and a SFRS with a slender aspect ratio. Although not explicitly expressed in the PEER (2017) guidelines; it is implied that the goals are aligned with those of ASCE 7-16 by invoking the "alternative design" method permitted by both ASCE 7 and the IBC. The overarching goals identified in ASCE 7-16, expressed as targeted maximum probabilities of structural collapse, P[C], given an MCE_R intensity level of ground shaking, are 10% for Risk Categories (RC) I and II, 5% for Risk Category III and 2.5% for Risk Category IV. It is noted in PEER (2017) that these acceptable collapse probabilities stem from the work reported in FEMA P695. Chapter 16 of ASCE 7-16 on NLRHA is specified as a function of the risk-targeted Maximum Considered Earthquake (MCE_R), whereas the traditional code design process specifies a design load intensity corresponding to 2/3 MCE_R, which is referred to as the Design Earthquake (DE). Other implicit objectives of seismic design under ASCE 7-16 are expressed in qualitative terms such as the ability to withstand the DE load without significant hazards to lives (life safety objective) and to withstand more frequent lower intensity earthquakes with limited damage. It is further noted in the PEER (2017) guidelines that the expected level of damage under service level intensity hazard may include minor cracking of concrete and yielding of steel in a limited number of structural elements but that the damage should be easily repairable and not compromise the ability of the building structure to withstand an earthquake at the MCE_R intensity.

ASCE 41-17 contains multiple performance objectives as a function of the targeted building performance and the building category for various levels of shaking.

No targeted maximum probabilities of structural collapse are identified in the National Building Code of Canada. However, NRCC (2017) notes in qualitative terms that the primary objective of seismic design addressed by the NBCC is to provide an acceptable level of safety. Under strong ground shaking (which is defined as a ground motion having a probability of exceedance of 2% in 50 years), it is expected that buildings in general may sustain substantial structural and non-structural damage but they should have sufficient strength to support gravity loads. Buildings of greater importance such as hospitals are required to be designed to higher loads and are expected to be functional post-earthquake. This is indirectly achieved by amplifying the design forces by an importance factor.

The PEER (2017) guidelines require evaluations at two levels of ground shaking: one under an intensity of shaking corresponding to a probability of exceedance of 50% in 30 years termed Service Level Earthquake (SLE) evaluation and a second evaluation at MCE_R. Chapter 16 of ASCE 7 requires evaluation only at MCE_R and the NBCC prescribes one evaluation with an intensity of shaking corresponding to a probability of exceedance of 2% in 50 years.

4.2 Ground Motion Characterization

An important aspect of NLRHA yet complex process, as noted in all the above noted publications, is the proper characterization of time history ground motions, which involves proper selection and scaling of ground motion histories to a compatible target response spectrum. It is suggested to select motions that are compatible with the tectonic regime with similar magnitude, fault distance, source mechanisms and site soil conditions.

Table 2: Selected summary of nonlinear response history analysis provisions

Response History Analysis Components	**Design / Assessment Method for NLRHA**			
	ASCE 7–16 Chapter 16 ASCE (2017a)	**ASCE 41-17 ASCE (2017b)**	**PEER TBI PBSD Guidelines PEER (2017)**	**NBCC 2015 NRCC (2015) & (2017)**
General				
Explicit goals	P[C] < 10%, 5% and 2.5% for RC I or II, RC III and IV, respectively	Varies based on targeted building performance	Intended to be at least equal to ASCE 7-16 by invoking alternative solution	Not specified
Ground motion hazard analysis	Probabilistic and deterministic	Probabilistic and deterministic	Probabilistic and deterministic	Probabilistic
Ground motion level	MCE_R	Multiple	SLE, MCE_R	2% in 50 yrs
Target spectrum	UHS or multiple spectra with risk adjustment	UHS with or without risk adjustment	UHS or multiple spectra with risk adjustment	UHS or multiple spectra
Ground motion selection (at maximum considered earthquake intensity)				
Number of motions	≥ 11 pairs per target spectrum	≥ 11 pairs per target spectrum	≥ 11 pairs total ≥ 5 records per source contributing more than 20% to hazard but not less than 11 total ≥ 5 records pulse or no pulse subsets for sites where pulse-type motions are considered	<u>Method A</u> ≥ 5 records per suite ≥ 11 records per source ≥ 11 records (total for all suites) <u>Method B1 or B2</u> ≥ 11 records for each scenario target spectrum
Scaling / modifications of motions to match target spectrum				
General approach	Amplitude scaling or spectral matching	Amplitude scaling or spectral matching	Amplitude scaling or spectral matching	Amplitude scaling or spectral matching
Amplitude scaling	Match records on average to target, enforce 90% suite floor	Match records on average to target, enforce 90% suite floor	Match records on average to target, enforce 90% suite floor	Match records on average to target, enforce 90% suite floor
Spectral matching	Match records to target considering a 110% suite floor	Match records to target considering a 110% suite floor Not to be used with Method 2,	Match records to target considering a 110% suite floor	Match each record on average to target, enforce 110% suite floor
Period range	min[0.2T, $T_{90\%}$] – 2.0T	$0.2T_{min}$ – max[1,1.5T_{max}] where T_{min} and T_{max} are the smallest and largest 1st mode periods for the two principal directions	min[0.2T, $T_{90\%}$] – 2.0T	min[0.15T,$T_{90\%}$] – max[2.0T, 1.5s]

Modeling and analysis				
Modeling parameters of nonlinear elements	Consistent with ASCE 41 or applicable laboratory test data	Based on experimental evidence	Use hysteretic models that adequately account for strength and stiffness deterioration under imposed deformations and cyclic loading; physical test data	Representative of actual nonlinear cyclic load test data
Design seismic demand	Use mean values; where unacceptable response is permitted, use 120% median value but not less than mean value of acceptable responses	Where component response is independent of direction of action, use mean of maximum absolute responses, where unacceptable response is permitted, use 120% median value but not less than mean value of acceptable responses	Use mean values; where unacceptable response is permitted, use 120% median value but not less than mean value of acceptable responses	Use mean value of all ground motions when only 1 suite is used When 2 or more suites are used – use largest of the mean of each suite where each suite has at least 11 ground motions; otherwise, use mean of the n highest values of all suites where n is the average number of records in all suites
Response metrics and acceptance criteria (at maximum considered earthquake intensity)				
Peak storey drifts	μ < twice code typical limit;	No limit specified	Suite mean of absolute peak transient story drifts ≤ 0.03	μ of analysis using lower-bound strength properties < typical code limit
Residual storey drifts	No limit specified	No limit specified	Suite mean of absolute residual drifts ≤ 0.01	No limit specified
Deformation-controlled actions	μ < ASCE 41 CP limit x 1.0/I_e, or Q_u < 0.3Q_{ne}/I_e for critical and 0.5Q_{ne}/I_e for non-critical	μ < limit specified for the targeted performance level as per Chapter 10 for concrete SFRSs	δ of any response history analysis to be less than the ultimate deformation capacity, δ_u, of the component; with some exceptions	μ of analysis using lower-bound strength properties < CSA limit x 1.0/I_E or ASCE 41 LS limit x 0.7/I_E in absence of CSA limit or test data x 0.5/I_E
Force-controlled actions	$\gamma I_e(Q_u - Q_{ns}) + Q_{ns} \le Q_e$ where γ is taken as 2.0, 1.5 and 1.0 for critical, ordinary and non-critical actions, respectively	$\gamma\chi(Q_{uf} - Q_g) + Q_g \le Q_{CL}$ where γ is taken as 1.3, 1.0 and 1.0 for critical, ordinary and non-critical actions, respectively	For well-defined yield mechanisms: $(1.2 + 0.2S_{MS})D + 1.0L + E_M \le \phi_s R_n$ $(0.9 - 0.2S_{MS})D + E_M \le \phi_s R_n$	μ of analysis using upper-bound strength properties < resistance determined from CSA standard

		and χ is taken as 1.0 for CP and 1.3 for LS and IO and $\gamma\chi \leq 1.5$	For other mechanisms: $(1.2 + 0.2S_{MS})D + 1.0L + 1.3I_e(Q_T\text{-}Q_{ns}) \leq \phi_s BR_n$ $(0.9 - 0.2S_{MS})D + 1.3I_e(Q_T\text{-}Q_{ns}) \leq \phi_s BR_n$	
Treatment of collapse or unacceptable response cases	1 unacceptable response permitted for RC I & II where spectral matching is not used	1 unacceptable response per 11 analyses permitted for LS or lower performance levels	Where ϕ_s is as per ACI 318 for critical elements, and 0.9 and 1.0 for ordinary and non-critical elements, respectively	Where spectral matching is not used, 1 unacceptable response permitted if suite has at least 11 ground motions and additional evaluations are carried out

The estimation of the ground shaking is typically done either deterministically or probabilistically. A probabilistic approach accounts for uncertainties by considering weighted alternatives of various earthquake sources, possible magnitudes, multiple viable ground attenuation relationships (ground motion prediction equations) and earthquake recurrence. The cumulative probabilities of all considered scenarios is then used to quantify the seismic hazard. In a deterministic approach, the seismic hazard is simply defined as the maximum response resulting from the considered scenarios.

Whereas ASCE 7-16 Chapter 16, PEER (2017) and ASCE 41-17 typically define the seismic hazard at the MCE_R intensity using a probabilistic approach with a deterministic cap, the NBCC hazard stems from a purely probabilistic approach. Unlike the US, there is no deterministic cap applied to the hazard assessment in Canada. PEER (2017) specifies the use of a probabilistic approach for the characterization of the Service Level Earthquake (SLE) Evaluation ground shaking hazard.

Selected ground motion histories are then scaled to match a target response spectrum to ensure adequacy of the adjusted histories for use in analysis. ASCE 7-16, PEER (2017) and NRCC (2017) indicate that the target spectrum can be based on a uniform hazard spectrum or a spectrum constructed from multiple scenarios.

Ground motion records can be either amplitude scaled or matched spectrally, over a defined period range, to a minimum specified level. The period range is typically specified to span from a minimum period $T_{90\%}$, which corresponds to the lowest period required to capture at least 90% of the mass participation to a maximum taken as 2 times the lateral fundamental period of the structure, T. An absolute minimum of 0.2T is specified for the range in PEER (2017) and ASCE 7-16. On the other hand, Commentary J of the NBC User's Guide (NRCC 2017) specifies an absolute minimum of 0.15s regardless of the $T_{90\%}$ value in determining the period range and a minimum upper range of 1.5s. In general terms, records are scaled over the period range such that the suite meets 90% and 110% of the targeted spectrum for amplitude scaling and spectral matching, respectively.

In addition to properly scaled ground motions, an adequate number of records is needed to properly predict and quantify the response of a structure subjected to ground shaking. PEER (2017) specifies the minimum number of ground motions to be used as a function of the type of assessment to be undertaken. For SLE evaluation, a minimum of 3 and 7 pairs of ground motion histories are specified for linear and nonlinear analyses, respectively. For MCE_R evaluation, a minimum of 5 records are to be used for each source contributing more than 20% to the hazard but not less than 11 records in total. A minimum of 5 pulse-type records is also specified for sites where pulse-type motions are to be considered. The minimum of 11 pairs at MCE_R evaluation aligns with ASCE 7 Chapter 16 requirements.

In Canada, the minimum number of records is expressed as a function of the method used to define the target response spectrum. Where Method A is used, minimum requirements specify 5 records per suite, 11 records per source and an absolute minimum of 11 records. Where Method B1 or B2 is used, a minimum of 11 records for each scenario target spectrum is specified.

Contrary to code-based approaches which only consider one level of shaking, ground motions used for PBSD are to be characterized for each level of shaking considered.

4.3 Modeling and Analysis

Proper modeling and analysis of building structures is another important aspect of NLRHA and performance-based seismic design. PEER (2017) recommends modeling of structural components based on expected properties to provide an unbiased estimate of the expected response of the building. It specifies the expected concrete strength as 1.3 times the specified concrete strength. It is also noted that the 1.3 multiplier may be different depending on the strength of the concrete, the use of fly ash and other additives as well as the type of aggregates used. NRCC (2017) specifies the use of lower-bound strength properties and upper-bound strength properties for modelling of non-linear SFRS elements to determine demands for deformation-controlled and force-controlled actions, respectively.

For modeling of nonlinear components, all publications indicate that proper material and hysteretic models that adequately account for strength and stiffness deterioration under imposed deformation and cyclic loading should be used. It is also inferred that model parameters should be validated by experimental data.

The PEER (2017) guidelines require the use of well-defined three-dimensional mathematical models which appropriately reflect the spatial distribution of the mass and the stiffness of the building structure. The models are also to be representative of the anticipated effective stiffness and damping under the level of shaking under consideration. The NBCC does not have any explicit requirements with respect to three-dimensional modeling for use with nonlinear time history analysis.

4.4 Acceptance Criteria

As a final step in response history analysis, the seismic demand is compared and checked against acceptance criteria. Requirements regarding the determination of the seismic demand vary depending on the level of evaluation under consideration and the type of analysis used. Under SLE, PEER (2017) indicates that typical traditional code-based response modification factors are not to be applied to response spectrum or linear history analysis results as the intent is to have the buildings remain essentially elastic under this level of shaking. Where nonlinear analysis is used, PEER (2017) generally specifies the use of average values to determine the seismic demand for both SLE and MCE_R evaluations. In Canada, Commentary J of the NBC User's Guide specifies that mean values of all ground motions may be used where only 1 suite is used. When 2 or more suites are used, the largest of the mean of each suite, where each suite has at least 11 ground motion may be used; otherwise, the mean of the n highest values of the response parameters (where n is the average number of ground motions in all suites) is to be used.

All publications identify unacceptable responses. In general, these include an analysis which fails to converge, cases where the predicted demand exceeds the range of modelling or capacity of the component and demands which exceed deformation limits at which members can no longer support gravity loads. One unacceptable response is permitted in some cases as highlighted in Table 2.

Response metrics and acceptance criteria are specified in terms of global and component acceptance criteria. Global acceptance criteria include drift limits which are further subcategorized into peak storey drifts and residual storey drifts. PEER (2017) specifies an overall peak storey drift limit of 0.005 for SLE evaluation and a mean absolute peak transient storey drift limit of 0.03 for each suite for the MCE_R evaluation. A mean absolute residual story drift limit of 0.01 is also specified for MCE_R evaluation. Peak storey drift limits for response time history analysis under ASCE 7-16 and NBCC 2015 are noted as twice the typical code drift limits and the typical code drift limits, respectively. These documents do not list any requirements/limits for residual storey drifts.

For component checks, all publications require the classification of actions into either deformation-controlled actions or force-controlled actions. Deformation-controlled actions are noted as those actions that are expected to undergo nonlinear behavior in response to earthquake shaking whereas as force-controlled actions are not expected to undergo nonlinear behavior. A second tier of classification (critical, ordinary and non-critical) further distinguishes the consequence of failure of the force-controlled actions in PEER (2017), ASCE (2017a) and ASCE (2017b). The acceptance criteria for deformation- and force-controlled actions are listed in Table 2. As noted, demands or acceptance criteria for these components are generally adjusted to reflect the importance category of the building under evaluation. For force-controlled actions, the US publications in general require that the portion of the seismic only demand be amplified to obtain the design demand for these components which is then compared to the resistance of the component using either expected or lower-bound material properties. In Canada, the demand for force-controlled components are to be computed based on the probable resistance of the SFRS or using an analysis which considers upper-bound strength properties for the nonlinear elements.

Given the complexities involved with nonlinear time response history analysis, all four documents require review by an independent third-party engineer with experience in nonlinear analysis and seismic design.

5 ADAPTATION TO CANADIAN CONTEXT AND NEED FOR FUTURE RESEARCH

As described in this paper, most of the documentation to date on performance-based seismic design has been developed for the assessment and retrofit of existing buildings. To the authors' knowledge, few guidelines exist (other than PEER 2017 and LATBSDC 2017) on PBSD of new buildings, with these being focused on tall buildings. Other than the limited information introduced in Commentary 'L' of the NBC User's Guide (NRCC 2017), the authors are unaware of any PBSD information that is specific to Canada. It is also noted that the NBCC does not quantify the target performance intended by the Code in terms of probability of collapse as does ASCE 7.

On that basis, the following is identified as needs for future research in Canada:

- elaboration / development of performance-based seismic design framework for assessment of existing buildings and design of new buildings (levels of ground shaking, performance levels, performance categories), and
- elaboration / development of nonlinear modelling parameters and acceptance criteria for use within a PBSD framework for various levels of shaking consistent with the minimum detailing requirements outlined in the respective CSA material design standard.

It is important to note that in Canada, the National Building Code of Canada prescribes design forces whereas the referenced CSA material design standard specifies the material resistance. In general, there is limited guidance provided with respect to the mathematical and computer modeling of building structures. This is a gap that also needs to be addressed, especially considering the availability of modeling software tools and the shift to dynamic analysis as the default method. Although the information stems from a US methodology perspective, the PEER (2017) guidelines and ASCE 41-17 provide valuable insights on this.

6 CONCLUSION

This paper explored the North American state-of-the practice in NLRHA and performance-based seismic design of buildings with an emphasis on reinforced concrete seismic force resisting systems. As noted, PBSD provides various stakeholder with a better representation of the expected performance of a building under earthquake shaking. Recent guidelines developed by PEER have also been explored. However, these guidelines strictly focus on the PBSD of tall buildings and stem from a US methodology perspective. As it can be deduced from the above noted literature review, Canada is significantly lagging the US in the development of Performance-Based Seismic Design methodology. Just as linear dynamic analysis is now the default method in current Canadian code-based seismic design, it is anticipated that performance-based seismic design will be the next generation seismic design methodology in the future.

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