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Determination of Seismic Response Modification Factors in Canada and the United States: A Critical Review

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ABSTRACT

The equivalent static force procedure is a fundamental part of seismic design provisions of the National Building Code of Canada (NBC). This procedure simplifies the seismic design process by approximating the complex nonlinear dynamic response of structures using an equivalent linear problem. Two key input parameters in the equivalent static force procedure are the ductility-related and overstrength-related force modification factors. The ductility-related factor accounts for the ability of a structure to dissipate energy through inelastic behaviour. The overstrength-related factor considers the reserve strength of a structure due to material properties and redundancy in the system. The seismic force modification factors of the NBC were originally determined based on engineering judgment and a qualitative approach. However, it is not clear whether this approach provides a uniform performance between the various types of structural systems. Additionally, implementation of new seismic-force-resisting systems (e.g., energy dissipation devices) into building codes requires a standardized procedure to quantify the seismic force modification factors. In the US, FEMA P695 provides a rational standard method to determine response parameters that are equivalent to the seismic force modification factors in NBC. A similar procedure needs to be developed for the Canadian codes to ensure consistent evaluation of seismic force modification factors. This study provides a historical review of the development of the seismic response modification factors in Canada and US to gain a better understanding of the rationale and appropriateness of the existing factors. The similarities and potential areas of inconsistency between the factors used in both building codes are discussed in detail. The findings of the study have contributed to the recent development of the performance-based unified procedure, a standardized method for determination of seismic force modification factors for structural systems in Canada.

Keywords: seismic design, equivalent static force procedure, seismic force modification factors, National Building Code of Canada, FEMA P695.

INTRODUCTION

Dynamic analysis of structures is a complex and time-consuming task that requires considerable expertise. Many structural design codes, including the National Building Code of Canada (NBC) [1], allow engineers to use an alternative analysis approach known as the Equivalent Static Force (ESF) procedure for seismic design of structures that meet certain requirements. This procedure is based on the concept of approximating the complex nonlinear dynamic response of structures with an equivalent linear problem. Because of its simplicity and effectiveness, ESF has become a fundamental part of seismic design provisions of NBC and has been commonly used in engineering offices. A key aspect of ESF is the reduction of seismic design force from the linear elastic demand level by the ductility- and overstrength-related force modification factors. The ductility-related force modification factor (R_d) accounts for the ability of a structure to dissipate energy through inelastic behaviour. The overstrength-related force modification factor (R_o) considers the reserve strength capacity of a properly designed, redundant structure which can come from various sources including overstrength in material properties relative to the nominal design values and inherent redundancy in structural systems. The two seismic force modification factors are critical in estimating the strength and deformation demands of structural systems that are expected to perform in an inelastic range during earthquakes, when using simplified linear methods such as the ESF design procedure.

The seismic force modification factors (R_d and R_o) were originally determined based on the engineering judgment and a qualitative approach which involved comparing the performance of several seismic-force-resisting systems (SFRSs) during past earthquakes. However, because of the lack of a standardized approach for the determination of these factors, it is not clear whether they result in consistent performance for different structural systems. Also, the existing factors are estimated based on the comparison of the performance of limited number of SFRSs under certain levels of earthquake ground motions, which raises questions about their range of applicability and their ability to represent the inelastic seismic performance of structures. More importantly, a large number of new seismic-force-resisting systems and high-performance structural systems (e.g. energy dissipation devices) have been developed in recent years and are increasingly becoming available for engineers to consider using in the seismic design of buildings. To properly address these new and innovative systems in design codes, there is a need to thoroughly investigate and quantify their ability to meet the intended seismic performance objectives including the life safety goal. This requires development of a standardized framework for determining the seismic force modification factors that is consistent with the existing approach and definition in the NBC. In the US, the FEMA P695 document entitled "Quantification of Building Seismic Performance Factors" [2] presents a rational standard procedure to determine building system performance and response parameters (which are equivalent to the seismic force modification factors in the Canadian code) for the linear design methods used in the American building codes. A similar procedure needs to be developed for the Canadian codes to establish consistent seismic force modification factors for various SFRSs designed and built based on the Canadian construction practice and seismic detailing.

Recently, the National Research Council of Canada (NRC), in collaboration with several Canadian universities, have developed a performance-based unified (PBU) procedure [3] that allows engineers to systematically quantify the seismic force modification factors for different SFRSs defined in the NBC. While the PBU procedure is mostly based on FEMA P695, it includes major modifications to make it consistent with the NBC seismic performance objectives and also enable evaluation of structural systems in a more computationally efficient way. The development of PBU procedure required identifying similarities and potential inconsistencies between the seismic response modification factors defined in the Canadian and American building codes. The applicability of FEMA P-695 to Canadian seismic design provisions also should be evaluated. This paper presents a summary of the work conducted by a research team from Carleton University and NRC to address these tasks. The paper first provides a historical review of the development of the seismic response modification factors in Canada and the US in order to gain a better understanding of the rationale and appropriateness of the existing modification factor values. The paper then presents a short summary of the P-695 procedure and identifies potential challenges that needs to be overcome in order to adapt and implement the methodology for the Canadian context.

HISTORICAL DEVELOPMENT OF THE SEISMIC RESPONSE MODIFICATION FACTORS

Seismic response modification factors for use in equivalent static force procedures are known by many different names in the different codes and standards around the world. They may be known as seismic performance factors, strength reduction factors or system response coefficients in the United States. These factors, which are also generally known as *R* factors (*R* stands for reduction), were first presented in the ATC 3-06 document entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings" published in 1978 [4]. The ATC 3-06 seismic design provisions resulted from a concerted effort of both the public and private sectors to improve the seismic performance of buildings subjected to extreme seismic events and reducing losses through improved design practices. This effort included design professionals, researchers, government regulatory agencies and building code organization representatives and staff.

Since then, in the past four decades, there has been a considerable amount of research in the US into the theoretical basis of the seismic response modification factors in terms of their effects on seismic demand and expected performance. This included research into the relationship between strength reduction, ductility and drift displacement [5-12]. The definition of the point of collapse in the inelastic response of a structure also directly influences the determination of these response parameters and their relationships to each other. Furthermore, since the first introduction of *R* factors to the American design codes, there have been substantial advancements in seismic design at the material, component, and system levels. In Canada, there has not been any comparable parallel development in the values of the seismic response modification factors. The *R* factors for some more recently developed seismic force resisting structural systems have been determined based on performance simulation and assessment using advanced seismic response numerical modelling and simulation tools, though not in a systematic way (e.g. buckling restrained brace frames [13]); however, many of the original *R* factors of some commonly used structural framing systems, such as steel and concrete moment-resisting frames, have largely remained the same since the time of their introduction decades ago. Those legacy *R* factor values were selected largely based on the consensus of experts and the state of knowledge and practice at the time rather than being based on any rigorous formulation and evaluation. Consequently, there is a lack of consistency in the values of the *R* factors among the current list of SFRSs in the building codes.

Table 1 presents a summary of the historical development of the seismic response modification factors together with their implementation in design guidelines and codes in the US and Canada. Overviews of the historical development of the seismic response modification factors have been presented elsewhere in ATC 19 [14] and by Malley and Cheever [15].

Table 1: Summary of development of seismic response modification factors in Canada and US

US	Canada
1959 SEAOC (SEAOC Blue Book) [16] – 10% in 50 years; The base shear for buildings in California was given as: $$V = KCW \qquad (1)$$ where K is the horizontal force factor (the predecessor of R factor), C is related to the fundamental period, W is the total dead load. K = 1.33, 0.80, 0.67, 1.00 for bearing wall, dual systems, moment-resisting frames and other frames, respectively.	
1961 UBC [17] – 10% in 50 years; Adopted 1959 SEAOC, but added seismic factor Z for seismic zonation outside California. $$V = ZKCW \qquad (2)$$	**1965 NBC [18]** – 10% in 50 years; First introduction of construction factor (C); The base shear was given as: $$V = RCIFSW \qquad (3)$$ where R is the seismic regionalization factor, I is the importance factor, F is the foundation factor, S is the structural flexibility factor, and W is the total weight. C is equal to 0.75 for ductile moment frames and shear walls, and 1.25 for all other structural systems.
1978 ATC 3-06 [4] - 10% in 50 years; First introduction of R factor. $$\mathrm{V} = \frac{1.2A_vS}{RT^{0.67}}W \leq \frac{2.5A_a}{R}W \qquad (4)$$ where A_a, A_v are seismic hazard parameters, S is a soil profile coefficient, and T is the fundamental period of building. The intent was to improve seismic performance by requiring better detailing rather than increasing the design base shear.	**1975 NBC [19]** – 10% in 50 years; The construction factor changed to K. Values of K for different structural systems were provided. $$V = ASKIFW \qquad (5)$$ where A is the design ground acceleration and S is the seismic response factor based on the fundamental period; I, F and W are as defined in 1965 NBC with some minor modifications. Stated that K=1/μ where μ=structural ductility
1985 NEHRP [20] – 10% in 50 years; Followed a strength design method; adopted R factor from ATC 3-06	
1988 SEAOC [21], 1988 UBC [22] – 10% in 50 years; Followed an allowable stress design method; adapted ATC 3-06 as R_w factor; the design base shear at the allowable stress level was: $$\mathrm{V}_D = \frac{ZICW}{R_w} \qquad (6)$$ where Z and I are the seismic zone and importance factors, respectively.	**1990 NBC [23]** – 10% in 50 years; Replacement of the K factor by the force modification factor R. $$V = \frac{U(vSIFW)}{R} \qquad (7)$$ where U is a calibration factor equal to 0.6, and v is the velocity zone ratio.
Current Design Code in the US	**Current Design Code in Canada**
ASCE 7-22 [24] – 2% in 50 years for MCE_R; Design earthquake = 2/3MCE_R $$V = \frac{S_aI_EW}{R} \qquad (8)$$ where S_a is the design spectral acceleration and I_E is the importance factor. For non-fuse elements (collectors, connections, etc.) V is multiplied by the overstrength factor Ω_0.	**NBC 2020 [1]** – 2% in 50 years (changed since NBC 2005). $$V = \frac{S(T_a)M_vI_E}{R_dR_o}W \qquad (9)$$ where R_d and R_o are ductility and strength force modification factors, M_v is the higher mode factor, and T_a is the fundamental period.

Note: Eqs. (8) and (9) were originally proposed in earlier editions of the code not provided in this table. The focus of this table is on the R factors and the history of changes made to other parameters of the base shear equation is not provided.

In ATC 3-06 [4], the values of *R* factors for various structural framing systems were selected through committee consensus on the basis of i) general observed performance of similar building systems during past earthquakes; ii) estimates of the general ability of the building framing type to absorb seismic energy and deform beyond the elastic limit without serious degradation in strength capacity; and iii) estimates of the amount of damping present during inelastic response.

The initial values of these *R* factors for typical building framing systems were derived from the values of their predecessor horizontal force *K* factors in the 1959 *Structural Engineers Association of California (SEAOC) Recommended Lateral Force Requirements*, commonly referred to as SEAOC Blue Book [16]. The *K* factor values for bearing wall buildings, dual systems, moment-resisting frames and other framing systems were 1.33, 0.80, 0.67 and 1.0, respectively. These horizontal force *K* factor values represented the consensus opinion of the experts (design professionals and researchers) based on the state of knowledge and practice of earthquake-resistant design at the time. A committee on structural design, details and quality assurance was tasked with the transition from the *K* factors to the new seismic response modification factor *R* which was moved from the numerator to the denominator in the base shear expression for the building seismic design. Following a direct translation process of the base shear expression given by 1976 UBC [25] and ATC 3-06 [4], a relation between the *K* factor used in 1976 UBC and the *R* factor adopted in ATC 3-06 (1978) was derived as follows:

$$R = \frac{5.1}{K} \tag{10}$$

In the derivation of Eq. 10, it was assumed that the allowable stress design had a margin of safety of 1.67 and a permissible increase of the allowable stress by a factor of 1.33 for the extreme earthquake load in the 1976 UBC, while the strength design in the ATC 3-06 design had a material reduction factor of 0.9. For example, for a steel moment-resisting frame building with K=0.67 in the 1976 UBC, a seismic response modification factor R=8 was obtained.

The seismic design provisions with the *R* factors in the ATC 3-06 [4] were adopted in the 1985 *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* [20] which was a strength design method. Because of the two parallel existing design approaches of strength design and allowable stress design for buildings at the time, the concept of response modification for seismic design was also adopted in the allowable design approach as the R_w factor in the 1988 SEAOC, and 1988 UBC. A relationship was derived between the *R* factor in ATC 3-06 1978 and the R_w factor in 1988 UBC [14, 15]:

$$R_w = 1.54R \tag{11}$$

In Canada, the fundamentals and conceptual approach for earthquake-resistant design have been generally similar to those described above; however, there are some important differences in the history and development of *R* factors here. This history is summarized below and contextualized to identify differences, similarities or overlap in the targeted performance level or quantification of seismic hazards between Canada and the US. Reviews of the evolution of the seismic design provisions in Canada have been presented in [26-29].

In Canada, the *K* factor predecessor to the *R* factor in the 1975 NBC [19] was construed as the reciprocal of the structural ductility factor μ. It is likely the values of structural ductility factor μ for the different structural framing systems were selected based on engineering judgement and consensus expert opinion. While there was information available of the development of the seismic response factors in the US, there is very little documented information available for the development and experiences in Canada. Mitchell et al. [26] provided a comprehensive history of the available information about the development of the seismic design provisions of buildings in Canada; however, there is no information about the selection of the design values of the *K* and later *R* factors. In the 1990 NBC [23], the base shear formula adopted the *R* factor format in conjunction with a calibration factor *U* in its expression. This calibration factor U = 0.6 was determined such that, for existing SFRS, the resulting *R* factor in the 1990 NBC resulted in approximately the same design forces as the previous 1975 NBC. This calibration *U* factor was interpreted as representing the overstrength capacity that allows buildings to resist seismic loads that are higher than the design seismic force [28]. This *U* factor was later moved to the denominator and became the overstrength factor R_o in the 2005 NBC [30]. The previous *R* factor became R_d. The R_o factor now explicitly accounts for the higher strength capacity of the building caused by restricted choices for sizes of members, rounding of sizes and dimensions, actual material yield being higher than the nominal design yield value, and strain hardening [27].

Before 2005, for seismic design of buildings the seismic hazard in Canada was considered at the probability of exceedance level of 10% in 50 years, equivalent to a 475 year return period earthquake. The US design codes used approximately the same hazard level. Since then, seismic hazards considered for seismic design of buildings in both Canada and US are specified at the probability of exceedance level of 2% in 50 years, equivalent to a 2475 year return period earthquake. The reason for adopting the lower probability was that it is closer to the probability of structural failure and provides more uniform safety margin against collapse [29]. It is worthwhile to note that in the US, the seismic hazard of 2% in 50 years specified in ASCE 7 for the risk-targeted maximum considered earthquake (MCE_R) has a targeted probability of structural failure at 1% in 50 years.

In the current edition of 2020 NBC [1], the *R* factors are expressed as the product R_dR_o of the ductility factor R_d and overstrength factor R_o. The current US building code ASCE 7-22 [24] has the seismic response modification factors expressed in terms of the ductility factor *R*, the displacement amplification factor C_d and the overstrength factor Ω_o. Figure 1 shows an inelastic force-deformation curve of an idealized building to illustrate the seismic response modification factors used in ASCE 7 and NBC. It is noted that the *R* factor for US building designs gives the same combined total effect of the ductility parameter R_d and the overstrength parameter R_o. Thus, the *R* factor in the US code is equivalent to the combined reduction effect of R_d and R_o in NBC. Since the influence of overstrength in the US code is already included in the *R* factor as a whole, the overstrength factor Ω_o is not used in determining the design base shear earthquake force of the building. The overstrength factor Ω_o is used in the capacity design process for adjacent non-ductile structural components and connections in the building to ensure the building will reach its target ductility level without premature failure of the protected components. The displacement amplification factor C_d starts equal to *R* but is adjusted to account for increased damping during inelastic response of the building. Therefore, the displacement amplification factor C_d in the US code is somewhat equivalent to the combined seismic response modification effect of R_dR_o in Canada (if the damping effect is neglected). An estimate of the total inelastic displacement of the building is obtained by multiplying the elastic displacement with the displacement amplification factor C_d in the US codes, whereas in the Canadian code the inelastic displacement is obtained by multiplying the elastic displacement with the combined seismic response modification parameter R_dR_o.

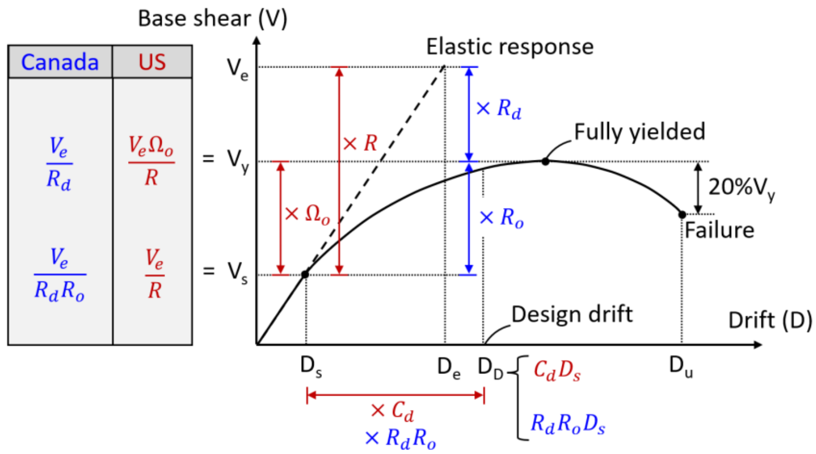

Figure 1. Seismic response modification factors in the US and Canada codes

COMPARISON OF *R* FACTORS AND DEFLECTION LIMITS IN CANADA AND US

When comparing sample values of the *R* factors between the US and Canadian building codes, it is important to note that there are considerably more detailed descriptions and specific requirements of the building structural framing system categories in the US codes than there are in the Canadian codes. In other words, the building structural framing categories in the Canadian codes are described in generally fewer specific terms and are thus applicable to a wider range of building structures. The significance and implication of this difference between the Canada and US codes needs to be further investigated.

The sample values of the *R* factors presented in Table 2 include historical figures to give a perspective of the evolution of development of the *R* factors and the state of knowledge and practice of earthquake-resistant design of buildings over the past three decades. There are two important issues that should be kept in mind when considering the historical evolution of the *R* factors and when comparing them between the US code and Canadian code. Frist, prior to the 2005 NBC [30], seismic design in Canada was based on using the seismic hazard level associated with a 10% in 50 years probability of exceedance. The input seismic hazard level was changed to 2% in 50 years in the 2005 NBC. Similar changes occurred also in the US building codes in the 1990s. The seismic hazard probabilities for each of the building codes listed in Table 2 are shown.

The second issue of importance for consideration when comparing the US and Canadian *R* factor values is that the *R* factor in ASCE 7-22 [24] is defined relative to the design forces that are associated with the *design earthquake* which is taken as 2/3 of the MCE_R seismic hazard (which has a 2% in 50 years probability of exceedance). In contrast, in the Canadian building code the *R* factors are defined directly relative to the maximum earthquake ground motion considered, which has a 2% in 50 years probability of exceedance. The *design earthquake* as defined in the US code is not used in the NBC in the seismic design procedure for buildings in Canada. Consequently, there is an important inconsistency between the US *R* factor values and the Canadian code *R* factor values. The 2/3 factor in the US code can be interpreted as approximately equivalent to scaling the 2% in 50 years seismic hazard to the hazard level of 10% in 50 years to determine the seismic design forces. Since that equivalent 10% in 50 years hazard of the *design earthquake* is approximately the same as the hazard level in previous old codes, the *R* factor in the ASCE 7-22 code can be directly compared with the *R* factor values in the old US codes presented in Table 2. A comparison of the *R* factor values of ASCE 7-22 with ATC 3-06 [4] and NEHRP 1991[31] shows that they are quite consistent overall with only small *R* factor changes for low-ductility or non-ductile framing systems. When comparing with the 1990 NBC [23] *R/U* factor values (before the change to 2% in 50 years), they are also quite similar; however, for newer codes, the 2/3 scale factor for the design earthquake in ASCE7-22 requires that for comparison of the *R* factor values, the ASCE 7-22 *R* factor values should be multiplied by 1.5 when comparing with the 2020 NBC combined R_dR_o values. This allows an equivalent comparison relative to the 2% in 50 years seismic hazard level. For this comparison, the *R* factor values in Tables 2 clearly show that there is a significant difference between the US and Canada *R* factor values for seismic design with the ASCE 7-22 1.5*R* values on average 1.9 times the 2020 NBC R_dR_o values. Based on this preliminary analysis, this could mean buildings in Canada are required to be designed with seismic forces 1.9 times higher than comparable buildings in the US. It is clear that the validity of this observation needs to be further investigated.

Considering this inconsistency from another perspective, considering the original formulation of the NBC overstrength factor: $R_o = 1/U = 1/0.6 = 1.67 \approx 1.5$, the R_o factor may be interpreted as being equivalent to the 2/3 scaling factor employed in the US code to scale the seismic hazard from 2% in 50 years to 10% in 50 years. While this scaling factor is the constant 2/3 value in ASCE 7-22, the R_o factor in NBC varies from the baseline value of 1.5, increasing or decreasing slightly dependent on the building framing system. As such, it is also sensible to directly compare the *R* factor value in ASCE 7-22 with the R_d factor in 2020 NBC. Comparison of the ASCE 7-22 *R* factor values with 2020 NBC R_d factor values give the same observation with the US code *R* factor values on average about 1.9 times that of the Canadian code R_d values.

In light of this discrepancy between the Canadian and US building code *R* factor values, three issues need to be clarified: 1) the need to look into any potential difference in the modelling of the seismic hazard in the US and Canada building codes; 2) any potential difference in the targeted performance level of the design building in the US and Canada codes, and 3) differences in the design and construction practice of structural systems in the US and Canada. First, even though the input seismic hazard considered for the seismic design of buildings in the US and Canada has the same 2% in 50 years probability of exceedance, there may be some systemic difference in the seismicity modelling. This could result in the US seismic hazard model giving consistently higher seismic demand than Canada's seismic hazard model, which could minimize the actual difference in the design forces between Canadian and US design requirements. Consequently, the building collapse margin ratio (CMR) in the FEMA P695 [2] is relevant with regard to its potential impact on the selection of the *R* factor values in the US. The differences in the design, detailing and construction practice of structural systems in the US and Canada can also result in some seismic force resisting systems being stronger than their Canadian equivalent. This also needs further investigation and clarification.

For specific comparison example, for ductile steel moment-resisting frames the *R* factor (*1.5R* for ASCE 7-22 and R_dR_o for 2020 NBC) values range between 6.7 to 12. Whereas for less ductile systems or systems with nominal ductility, the range is between 2.6 to 5.25 and the range of difference is relatively larger for the less ductile framing systems compared to ductile design. Similarly, for concrete ductile moment-resisting frames, the corresponding ranges are 6.8 to 12, and 1.95 to 4.5, respectively, with the same trend of observation. Comparing between the values of *R* factors between the current codes in US and Canada, the same trend of difference in the *R* factor values is observed. The difference between the Canada and US code values for the *R* factors is higher for less ductile structural framing systems. For ductile concrete shear walls, the *R* factor values in the Canada and US codes are 5.6 and 7.5, respectively. Whereas for moderately ductile concrete shear wall in the Canadian code compared to ordinary reinforced concrete shear wall in the American code, they are 2.8 and 6.0, respectively. The implication of the bigger difference and lower Canadian code values of the *R* factors for less ductile structural systems is that, since many of the older existing buildings are of the less ductile category, the lower *R* factor would require these older less ductile buildings be designed for a higher seismic force when undergoing seismic retrofit potentially increasing the costs in Canada.

In evaluation of the seismic performance of buildings, the storey drift of the building is closely related to the damage state of the building. It is therefore useful to compare the code specified storey drift limits for building structures in Canada and US codes too. Table 3 shows the storey drift limits for buildings of different importance categories and equivalences in the US code. The smaller storey drift limits specified in the US code imply that for buildings with design strength, buildings in the US can be expected to suffer less damage compared to similar buildings in Canada. Alternately, the lower drift limits may require buildings in the US to be designed with a higher strength capacity in order to meet the lower limits resulting in stronger design.

Table 2: Comparison of R factors of common structural systems in Canada and US

Structural System	US				Canada			
	R ATC 3-06 1978 10% 50 yrs	*R* NEHRP 1991 10% 50 yrs	*R* ASCE 7-22 2%* 50 yrs	1.5*R* ASCE 7-22 2% 50 yrs	*R/U* NBC 1995 *U*=0.6 10% 50 yrs	R_d NBC 2020 2% 50 yrs	R_o NBC 2020 2% 50 yrs	R_dR_o NBC 2020 2% 50 yrs
Shear Walls								
Concrete shear wall (nominal ductility)	4.5	4.5			3.3			
Special reinforced concrete shear wall			5.0	7.5				
Ductile concrete shear wall					5.8	3.5	1.6	5.6
Ordinary reinforced concrete shear wall			4.0	6.0				
Moderately ductile concrete shear wall						2.0	1.4	2.8
Masonry shear wall (nominal ductility)	3.5	3.5			3.3			
Special reinforced masonry shear wall			5.0	7.5				
Ductile masonry shear wall						3.0	1.5	4.5
Braced Frames								
Steel eccentric braced frame		7.0-8.0	8.0	12.0	6.7	4.0	1.5	6.0
Concentric braced frame		7.0						
Steel ductile braced frame					5.0			
Steel special concentric braced frame			6.0	9.0				
Steel moderately ductile concentric braced frame						3.0	1.3	3.9
Moment-Resisting Frames								
Steel special/ductile moment-resisting space frame	8.0	8.0	8.0	12.0	6.7	5.0	1.5	7.5
Steel intermediate/moderately ductile moment-resisting space frame			4.5	6.75		3.5	1.5	5.25
Steel ordinary/nominal ductility moment-resisting space frame	4.2	4.5	3.5	5.25	5.0	2.0	1.3	2.6
Concrete special/ductile moment-resisting space frame	7.0	8.0	8.0	12.0	6.7	4.0	1.7	6.8
Concrete intermediate /moderately ductile moment-resisting space frame		4.0	5.0	7.5		2.5	1.4	3.5
Concrete ordinary/nominal ductility moment-resisting space frame	2.0	2.0	3.0	4.5	3.3			
Concrete moment-resisting space frame of conventional construction						1.5	1.3	1.95

* In ASCE 7, the MCE_R has a 2% in 50 years probability of exceedance which is approximately equivalent to 10% in 50 years probability of exceedance for the design earthquake.

Table 3: Storey drift limits in Canada and US codes

NBC 2020		ASCE 7-22	
Building Categories	Drift Limits	Risk Categories	Drift Limits
Low	2.5%	I	2.0%
Normal	2.5%	II	2.0%
High	2.0%	III	1.5%
Post-disaster	1.0%	IV	1.0%

A SUMMARY OF FEMA P695 METHODOLOGY

The FEMA P695 [2] methodology is based on the nonlinear static and dynamic time history analysis of various *archetype* building structures which are intended to represent a sufficient range of potential design possibilities for a given seismic force resisting system type to effectively characterize its behaviour. Based on the results of these analyses, subjected to a suite of earthquake time histories, the methodology provides appropriate values for the system response coefficients R, C_d and Ω_0. These are determined based on criteria for how far those buildings are from the point of collapse when they are subjected to the maximum considered earthquake (MCE) seismic hazard levels.

In contrast to the definition of the system response coefficients provided by the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* [20], FEMA P695 defines R, C_d and Ω_0 in terms of spectral coordinates for acceleration and displacement, instead of force and deformation. This is so that they are compatible with an incremental dynamic analysis approach. These definitions are summarized in **Error! Reference source not found.**. In the figure, S_{MT} is the spectral acceleration associated with the MCE level earthquake at the fundamental period of the structural system, SD_{MT} is the spectral displacement associated with the same point, S_{CT} is the spectral acceleration associated with collapse of the structural system, SD_{CT} is the spectral displacement associated with the same point, S_{max} is the maximum strength of the structural system divided by effective seismic weight, and C_s is the seismic response coefficient which represents the design-level acceleration.

Error! Reference source not found. shows that R and C_d are defined in terms of the MCE level earthquake hazard. This is another difference from the NEHRP Recommended Provisions, which defines R relative to the design-based earthquake (DBE) hazard level. In practice, these two are equivalent since the ratio of the MCE acceleration S_{MT} to the design acceleration C_s is given as 1.5R. This reflects the fact that the MCE to DBE hazard levels also differ by a factor of 1.5. This differs significantly from Canadian practice, which defines response parameters relative to the MCE level hazard directly (without any 1.5 factor on the R equivalent).

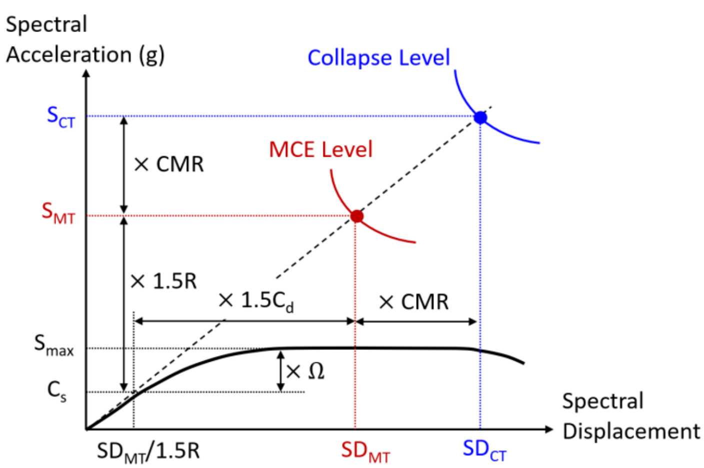

Figure 2. Illustration of seismic performance factors as defined by the FEMA P695 methodology [2]

The required performance level for determination of the system response coefficients in FEMA P695 is *life safety*, which is consistent with both the NBC and ASCE 7. This is in contrast to a performance objective of *collapse prevention*, which would imply that the building designs at the MCE seismic hazard level would be just on the verge of collapse. In this case, *life safety* performance is quantified using a calibrated collapse margin ratio (CMR). The CMR is a ratio of the seismic demand which causes half of the models to collapse at the MCE seismic demand level, i.e. the median collapse level. The acceptable CMR is a function of the required probability of collapse at the MCE level as well as the uncertainty in the determination of the median collapse level. The CMR is not a concept that is found currently in Canadian seismic design.

The methodology is intended for use in new building structures in the United States and requires accordingly that designs of these building structures adhere to the requirements of ASCE 7. It also requires that the designs adhere to the applicable structural and material standards for seismic design in steel, concrete, etc. There are quite a few important differences between the ASCE 7 approach to seismic design and the approach in the NBC. One that is particularly important to the methodology is that in ASCE 7, seismic design requirements for different types of structural system (applicability, height limits) are governed by the seismic design categories (SDCs), and in FEMA P695 the SDC is suggested as one of the parameters to use in defining building archetypes. This SDC is defined based on a risk category and seismicity of the building site. In Canada, the 2020 NBC introduces four seismic categories (SC) defined based on the importance factor of the building and the seismic hazard of the site. The SC is used to specify restrictions for the application range of the equivalent static force procedure and the usage of different structural systems based on the building height. Thus, there are differences in terms of the definition and application of seismic categories in NBC and ASCE 7. Prior to the 2020 NBC there were no seismic categories in the Canadian code and instead hazard ranges were used to specify seismic design requirements.

Before starting a FEMA P695 analysis for a given type of seismic force resisting system, there should be existing information about the behaviour of the system and the required detailing for implementation of the system to the level that it can be adequately represented by a nonlinear time history analysis model. This requires previous physical testing to acquire data on the behaviour of a sufficient range of system configurations and strengths. Assuming that the behaviour information is available first-hand or in the literature, the general steps required for a FEMA P695 analysis are [2]:

1. **Assume/propose a set of trial seismic performance factors (R, C_d and Ω_0) for the system.** This should be based on engineering judgement and the factors that are currently used for similar systems. This assumption will be checked for suitability at the end of the process.
2. **Develop a suite of archetypal building structures using the seismic force resisting system** considering variation in building height, period, framing configuration, bay width, gravity loading, and connection design and detailing. It is acknowledged that there is necessarily a limited number of possible architypes that can be considered. The methodology requires that initially a full design space of possible configurations should be considered which are then pared down to a minimum of four performance groups (two seismic design levels and two fundamental period ranges) which each contain typically three to six different architypes (typically resulting in 12 to 24 total building designs).
3. **Design and develop nonlinear structural models of each architype building.** Each architype building requires a full design for the seismic force resisting system using ASCE 7 and the applicable material design standards. This is typically done using the equivalent lateral force procedure or the response spectrum analysis procedure depending on the system. The nonlinear models should be calibrated to test data and be sufficiently sophisticated to be able to represent collapse of the structure (or at least represent phenomena that would trigger collapse).
4. **Conduct nonlinear static pushover analyses to determine overstrength Ω for each architype building.** Also, determine the period-based ductility, μ_T, which is a ratio of the ultimate yield displacement (at a 20% loss of strength past the peak), to the effective yield roof drift, which is a function of the fundamental period and the first mode shape of the building model.
5. **Conduct nonlinear dynamic time-history analyses to determine the median collapse point of each architype building.** This is done using a suite of 22 curated far-field strong motion earthquake records (each with x- and y-horizontal acceleration components). The median collapse point is calculated using a truncated incremental dynamic analysis procedure [32] where the records are all scaled up starting at the MCE hazard level and incrementing in steps until half of the earthquake records cause the architype to exceed the collapse criteria. This is estimated to require approximately 200 separate analyses for each architype building model.
6. **Determine appropriate system overstrength Ω_0 and determine suitability of trial response modification coefficient R.** Based on the results from nonlinear static pushover analysis, the value of the system overstrength factor Ω_0 is selected as the largest average value of Ω from each performance group. The suitability of the trial response modification coefficient R (selected in step 1 above) is evaluated based on calculated collapse margin ratios (CMRs) determined based on the nonlinear time-history analysis results. These are evaluated against two requirements: (1) the probability of collapse is less than 20% for each individual archetype, and (2) the probability of collapse is less than 10% on average for each performance group. The acceptable collapse margin ratio is also adjusted to take into account the uncertainty in the modelling, design requirements, system test data, and earthquake records. The deflection amplification factor C_d is the same as R but adjusted to account for system damping.
7. **If the response modification coefficient R is not suitable, select a new value,** redesign the architype buildings, and re-analyze to assess the suitability of the new selected R factor.

8. **Documentation and peer review are embedded throughout the process.** This is especially important since many of the steps above require a significant amount of engineering judgement.

The requirements for determining whether a building model has collapsed are not well-defined in FEMA P695. In the document, it states that the user should "*Process results to check for simulated collapse (lateral dynamic instability or excessive lateral deformations signaling a sidesway collapse mechanism) or non-simulated collapse (demands that exceed a certain component limit state criteria applied external to the analyses).*" [2]. The document gives some guidance about modelling to include stiffness and strength degradation, a simplified 20% strength reduction criteria for collapse, and a general definition of what a non-simulated collapse criterion may look like; however, the document does not contain many specifics about how to select or define collapse criteria.

FEMA P695 provides two earthquake suites to be used for time-history analysis as part of the methodology, a near-field suite (28 pairs) and a far-field suite (22 pairs). In the normal process of applying the methodology, only the far field set is used. The far-field suite includes strong motion records from around the world with different site types, and fault mechanisms. It does not contain any simulated records. It also does not contain any records from Canadian sites (there are no available strong motion records for Canadian sites). It will be necessary to evaluate if the existing earthquake record suite will be satisfactory to use in a Canadian context.

The scaling method used in P695 is not the state-of-the-art; however, it is simple, which is an advantage when so many scaling and analysis steps are required. The methodology scales the records in a two-step process. First, the records set is scaled to normalize their peak ground velocity. Then, the records are scaled directly as a set by the median spectral acceleration at the fundamental period of each building archetype as required by the methodology. This method is not consistent with the earthquake scaling method recommended in the NBC code structural commentary [33].

CANADIAN IMPLEMENTATION CHALLENGES

Based on the review of the FEMA P695 methodology and comparison of the seismic design process in the US and Canada codes, the study has identified some challenges for implementation of a FEMA P695 type analysis in a Canadian context. These are summarized below:

1. FEMA P695 requires that building archetype designs adhere to ASCE7, which has several aspects that are not found in the Canadian equivalent NBC:
    a. ASCE7 considers both a design earthquake hazard level and a maximum considered earthquake (MCE) hazard level. NBC only considers design at the MCE level.
    b. The definitions of the ASCE 7 seismic response factors (R, C_d, Ω_0) are not directly equivalent to NBC seismic response factors (R_d, R_o).
    c. ASCE 7 seismic response factors (R, C_d, Ω_0), are calibrated relative to the DE level. NBC seismic response factors (R_d, R_0) are calibrated relative to the MCE level.
    d. ASCE design relies on the definition of a seismic design category (SDC). FEMA P695 also references SDC in the definition of performance groups. The 2020 NBC, for the first time, introduced seismic categories (SC) to the Canadian code. However, the definition and application of SC in NBC are different than SDC in ASCE 7.
2. In FEMA P695, CMRs are increased by a spectral shape factor (SSF) to account for the fact that in the Western united states, the worst-case earthquakes have spectra which are concentrated around a small period range. The definition of the SSF factor is defined based on US-specific earthquakes, which may not be applicable in the Canadian context.
3. Collapse criteria are not specified in detail in FEMA P695, based on the literature, it is possible to make some more detailed recommendations to users of a Canadian methodology.
4. Earthquake record sets need to be evaluated for their suitability for use in the Canadian context.
5. Earthquake scaling methods in FEMA P695 should be evaluated so that they are in line with recommendations given in the NBC.

SUMMARY AND CONCLUSIONS

Seismic response modification factors (*R* factors) are key to the equivalent static force procedure, which is widely used in practice to design earthquake resistant buildings. This paper summarized the history and current state of *R* factors in the US and Canada. The inconsistencies and differences in *R* factors defined in structural codes of the two countries were identified and recommendations for future investigations were provided. It was shown that comparison of the *R* factors in ASCE 7-22 and 2020 NBC requires consideration of several factors including the variety and differences in specifications of seismic force resisting systems defined in the two codes, differences in terms of the basis of the definition of *R* factors in the two codes (the design earthquake versus the maximum considered earthquake), and also ductility of the structural system. The study found that in order to consistently compare the *R* factors between the two codes, the ASCE 7-22 *R* factor should be multiplied by 1.5 to account for the fact that the seismic design force in ASCE 7-22 is calculated based on 2/3 of the MCE_R, while the seismic design force in 2020 NBC is defined directly relative to the maximum earthquake ground motion considered (both ASCE 7-22 and 2020 NBC have the same probability of hazard). It was shown that there is a significant difference between the US and Canada *R* factor values for seismic design with the ASCE 7-22 *1.5R* values on average are 1.9 times the 2020 NBC R_dR_o values. To understand the reason behind this discrepancy any potential differences in the modelling of the seismic hazard and the targeted performance level of structures in the design codes as well as differences in the design, detailing and construction practice of structural systems in the US and Canada building codes should be investigated. The comparison of design codes also demonstrated that generally there is a larger difference between the *R* factor values of the 2020 NBC and ASCE 7-22 for structural systems with lower ductility.

The study also provided a review of the FEMA P695 methodology with a focus on identifying concepts and techniques in the methodology which are not currently found in Canadian seismic codes and standards. The main challenges for implementing the methodology into the Canadian context were identified as 1) addressing the difference in the hazard level used in NBC and ASCE 7 for determining the seismic design force, 2) finding equivalent concepts for the aspects of ASCE 7 and P695 that are not available in NBC (e.g., spectral shape factor), 3) developing a set of clear and practical criteria for determination of structural collapse, and 4) developing standard ground motion record sets suitable for different Canadian sites.

While the findings of this study will help engineers to gain a better understanding of the rationale and appropriateness of the existing *R* factors in the US and Canada, there is a need for a more systematic study to quantitatively evaluate the seismic performance of structural systems designed based on the American and Canadian structural codes using relevant *R* factors specified in ASCE 7-22 and 2020 NBC. By addressing some of the challenges specified in this study, the research team has recently developed a performance-based unified (PBU) procedure [3] comparable to the FEMA P695 methodology for implementation in Canadian codes that enables determination of R_d and R_o for different SFRSs in NBC in a systematic and consistent way.

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