Level 1: Preliminary Seismic Risk Screening Tool (PST) for existing buildings. Part 2: supporting technical documentation

Fathi-Fazl, Reza; Cai, Zhen; Jacques, Eric; Kadhom, Bessam

Client Report (National Research Council of Canada. Construction Research Centre); no. A1-013766, 2020-03-20

NRC·CNRC

Level 1 – Preliminary Seismic Risk Screening Tool (PST) for Existing Buildings

Part 2: Supporting Technical Documentation

Reza Fathi-Fazl, Zhen Cai, Eric Jacques, and Bessam Kadhom

Published by:

Construction Research Centre

National Research Council Canada | Conseil national de recherches Canada

Canada

This page is intentionally left blank.

LEVEL 1 – PRELIMINARY SEISMIC RISK SCREENING TOOL (PST) FOR EXISTING BUILDINGS

PART 2: SUPPORTING TECHNICAL DOCUMENTATION

Prepared by:

Civil Engineering Infrastructure Unit
Construction Research Centre
National Research Council Canada
Ottawa

Funded by:

Public Services and Procurement Canada

March 2020
Report No. A1-010575

This page is intentionally left blank.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the financial support from Public Services and Procurement Canada (PSPC), which led to development of the *Level 1 – Preliminary Seismic Risk Screening Tool (PST)*. Support from Parviz Afrooz, Brian Boyd, Peter Campbell, Bruno Coté, Dextor Edwards, Simon Foo, Clive Kamichaitis, Jocelyn Paquette, Doug Stephenson, Jack Vandenberg, Dave Weidelich, and Andrew Werblinski is also gratefully acknowledged.

The authors appreciate the effort made by Professor Dariush Motazedian from the Department of Earth Science at Carleton University in developing the site seismic categories for the *Level 1 – PST*. The authors acknowledge the contributions by Professor Ghasan Doudak from the Department of Civil Engineering at the University of Ottawa. Professor Doudak provided technical comments and inputs regarding definitions and descriptions of wood buildings.

The authors appreciate the editorial and technical feedback provided by Zoubir Lounis, Leonardo Cortés-Puentes, and Daniel Cusson from the NRC Construction Research Centre and the technical advices provided by Jitender Singh from the NRC Codes Canada.

This page is intentionally left blank.

TABLE OF CONTENTS

This page is intentionally left blank.

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 Background

There are thousands of existing buildings in Canada that could potentially suffer severe damage or collapse in the event of strong ground shaking. Assessing and mitigating seismic risk in large portfolios of existing buildings present technical and economic challenges to building owners. To address these challenges, the National Research Council Canada (NRC) developed a series of manuals and technical guidelines for seismic screening (NRC, 1993a), evaluation (NRC, 1993b), and upgrading (NRC, 1995) of existing buildings, based on the 1990 edition of the National Building Code of Canada (NBC 1990). The NRC screening manual (NRC, 1993a) was specifically developed to provide a quick and inexpensive screening procedure to identify and rank Canadian buildings in an inventory for further seismic evaluation. In 2001, Public Services and Procurement Canada (PSPC) issued the Real Property Services (RPS) Policy, which referred to the three aforementioned NRC technical guidelines. The RPS policy provides a seismic risk management approach for existing PSPC buildings.

The existing NRC technical guidelines should capture the current seismic requirements in the NBC as well as recent developments in seismic screening. The seismic code requirements in the 2015 edition of the NBC (NBC 2015) are significantly more stringent than those in the NBC 1990, on which the NRC technical guidelines and PSPC RPS Policy were based. Moreover, new methodologies for seismic screening, evaluation and retrofitting of existing buildings in the U.S. and around the world have emerged based on new data and research. A review of the state of practice and art of seismic risk screening of existing buildings is provided in APPENDIX A.

To update the existing PSPC seismic risk management approach, the NRC developed a multi-criteria and multi-level seismic risk management framework (Lounis et al., 2016). The framework consists of three key levels:

Level 1 – PST: Preliminary Seismic Risk Screening Tool (PST);

Level 2 – SQST: Semi-Quantitative Seismic Risk Screening Tool (SQST); and

Level 3 – SEG: Seismic Evaluation Guidelines.

Level 1 – PST and *Level 2 – SQST* correspond to the first and second volumes of the framework. *Level 1 – PST* is published in Volume I: Level 1 – Preliminary Seismic Risk Screening Tool (PST) for Existing Buildings, while *Level 2 – SQST* is published in Volume II: Level 2 – Semi-Quantitative Seismic Risk Screening Tool (SQST) for Existing Buildings. Each Volume consists of two parts: Part 1: User's Guide and Part 2: Supporting Technical Documentation. Part 2 of Volume I is provided in this document. *Level 3 – SEG* is under development and will be included in Volume III of the framework.

1.2 Intent

This document, *Level 1 – PST,* Part 2: Supporting Technical Documentation, complements Part 1: User's Guide. It provides details for developing the *seismic risk* screening procedure in the *Level 1 – PST*. Specifically, it describes the key *seismic risk* acceptance criteria, namely (1) *seismicity*, (2) *benchmark NBC edition*, (3) *remaining occupancy time*, and (4) *consequence of failure*. Furthermore, it includes the evaluation of these key criteria using *Level 2 – SQST* (Volume II) and HAZUS criteria (FEMA, 2003).

1.3 Organization

Chapter 2 presents and discusses the *seismic risk* acceptance criteria in *Level 1 – PST*, including the special conditions that immediately trigger *Level 3 – SEG*. Chapter 3 validates the *seismic risk* acceptance criteria by using the *Level 2 – SQST*. Chapter 4 provides details regarding the implementation of the *Level 1 – PST* to seventeen (17) *existing buildings* provided by PSPC.

2.0 DEVELOPMENT OF SEISMIC RISK ACCEPTANCE CRITERIA

This chapter provides the underlying methodology for establishing the *seismic risk* acceptance criteria used to assess if existing PSPC *buildings* covered by Part 4 of the National Building Code of Canada (NBC) are acceptable. In addition, special conditions that immediately trigger *Level 3 – SEG* are discussed.

2.1 Seismicity

Six *site seismic categories* (SSCs) – Very Low (SSC-0), Low (SSC-1), Moderate (SSC-2), Moderately High (SSC-3), High (SSC-5), and Very High (SSC-5) – have been developed to describe the lowest to highest seismicity across Canada. Development of the *site seismic categories* (SSCs) was a joint research effort between the NRC and the Department of Earth Science at Carleton University. The term "*site seismic category*" is adopted to distinguish it from the term "*seismic zone*" used in 1993 NRC screening manual. "Site" is included in the term because site conditions are incorporated in the seismic categorization. The following sections describe the rationale for developing the six *site seismic categories*.

2.1.1 MMI scale and anticipated building damage level

The Modified Mercalli Intensity (MMI) scale has long been used in seismic design practice to associate the perceived level of shaking with anticipated building damage. A coarse relationship between the anticipated building damage level and the MMI scale is presented in Table 2.1 (ASCE, 2016). MMI VI is generally considered to be the threshold for non-structural component damage, and MMI VII is considered to be the threshold for structural damage (Onur *et al.*, 2008). Therefore, the MMI scale with damage level is a useful benchmark for identifying regions of low *seismicity* and can be used to exempt *existing buildings* from *Level 2 – SQST*.

Table 2.1: Relationship between anticipated *building damage* and MMI scale

Anticipated *building damage* level	**MMI scale**
No *damage*	V
Light non-structural *damage*	VI
Hazardous non-structural *damage* Light structural *damage*	VII
Hazardous *damage* to susceptible structures	VIII
Hazardous *damage* to robust structures	IX+

The definition of low *seismicity* should be linked to the perceived level of shaking that corresponds to MMI level V and VI, for which no *damage* to structural components and no to light *damage* to non-structural components is expected.

2.1.2 Relationship between MMI scale and peak ground motion parameters

The MMI scale is qualitative and correlated to the anticipated *building damage* level. Several researchers have proposed empirical relationships between the MMI scale and peak ground motion (PGM) parameters. PGA and PGV are the primary PGM parameters used to delineate MMI *damage* levels, since they can easily be obtained from unprocessed accelerograms. A brief summary of some key studies are presented in this regard.

Wald *et al.* (1999) characterized relationships between MMI, PGA (in cm/s/s) and PGV (in cm/s) values, which are used for USGS ShakeMap program definitions in the ShakeMap Manual (Wald *et al.*, 2006), as shown in Table 2.2. They found a good fit with Eq. (2.1) for an MMI range of V to VIII, and with Eq. (2.2) for the IV to IX range. These relationships have been used widely in ShakeMap programs throughout the world. Note that log is the common logarithm.

$$MMI = 3.66 \times \log(PGA) - 1.66, \quad \text{for } V \leq MMI \leq VIII \tag{2.1}$$

$$MMI = 3.47 \times \log(PGV) - 2.35, \quad \text{for } IV \leq MMI \leq IX \tag{2.2}$$

Table 2.2: MMI damage level thresholds (Wald *et al.*, 1999)

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	*<.17*	*.17-1.4*	*1.4-3.9*	*3.9-9.2*	*9.2-18*	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	*8.1-16*	*16-31*	*31-60*	*60-116*	*>116*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Atkinson and Kaka (2004) studied the relationship between MMI and PGV values (instrumentally recorded and inferred form historical reports) for 18 earthquakes that occurred in eastern North America. They compared their results with the relationships presented by Wald et al. (1999) and concluded that the empirical relationships in eastern North America are different than in California. These differences are due to differences in the stress drop, regional attenuation, fault sizes, etc. Detailed discussion of the differences can be found elsewhere (Atkinson and Morrison, 2009). Atkinson and Kaka established the relationship between PGV and MMI as follows:

$$MMI = 3.96 + 1.79 \log(PGV) \tag{2.3}$$

$$MMI = (3.96 - 0.12) + 1.79 \log(PGV) - 0.28 \log(D) \tag{2.4}$$

where PGV is the peak ground velocity in cm/s and D is the source-to-site distance in km.

Note that the relationship proposed by Atkinson and Kaka (Eq. 2.4) is distance (D) dependent and it is not applicable to Appendix C of NBC 2015; however, for comparison, Atkinson and Kaka proposed relationships independent of distance, based on D = 1 km, D = 100 km, and D = 300 km.

Atkinson and Kaka (2007) determined relationships between ground motion and MMI that are applicable to earthquakes in the Central United States (CUS) by using data from felt moderate earthquakes in the CUS and in the New Madrid region for ShakeMap Program. They calibrated their data based on observations in California. They established the following relationship:

$$MMI = 4.40 + 1.92 \times \log(PGV) + 0.280 \times (\log(PGV))^2 \qquad (2.5)$$

where PGV is the peak ground velocity in cm/s, applicable both in the New Madrid region and in California, with a standard deviation of 0.78 MMI units.

Worden *et al.* (2012) updated the relationships established by Wald *et al.* (1999) and determined probabilistic relationships between ground-motion parameters and MMI in California, based on approximately 200,000 MMI observations. The earthquakes in their study ranged in magnitude from 3.0 to 7.3, and the distances from the source ranged from less than a kilometer to approximately 400 km. For intensities MMI ≥ 5, their results are in close agreement with the relations proposed by Wald *et al.* (1999). For lower intensities, their results fall midway between the relationships proposed by Wald *et al.* (1999) and those proposed by Atkinson and Kaka (2007).

$$MMI = c_1 + c_2 \log(Y), \quad \text{for } \log(Y) \leq t_1 \qquad (2.6)$$

$$MMI = c_3 + c_4 \log(Y), \quad \text{for } \log(Y) > t_1 \qquad (2.7)$$

where Y is the ground-motion amplitude of the selected type, c_1 through c_4 are the fitted parameters, and t_1 is the $\log(Y)$ condition. Table 2.3 provides the coefficients for the above equations and the standard deviations of the residuals. Table 2.4 shows the relationship between peak ground motions and MMI scales, which is proposed by Worden *et al.* (2012) and used in the ShakeMap program. Note that the values in Table 2.4 are thresholds for each of MMI scales.

Table 2.3: Coefficients for relationship between ground motion and MMI (Worden *et al.*, 2012)

Parameter	c_1	c_2	c_3	c_4	t_1 [$\log(PGM)$]	t_2 (MMI)	σ_{MMI}	$\sigma_{\log Y}$
PGA	1.78	1.55	-1.60	3.70	1.57	4.22	0.73	0.39
PGV	3.78	1.47	2.89	3.16	0.53	4.56	0.65	0.40
PSA 0.3 s	1.26	1.69	-4.15	4.14	2.21	4.99	0.84	0.46
PSA 1.0 s	2.50	1.51	0.20	2.90	1.65	4.98	0.80	0.51
PSA 3.0 s	3.81	1.17	1.99	3.01	0.99	4.96	0.95	0.69

Table 2.4: MMI damage level thresholds (Worden *et al.*, 2012)

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.05	0.3	2.8	6.2	12	22	40	75	>139
PEAK VEL.(cm/s)	*<0.02*	*0.1*	*1.4*	*4.7*	*9.6*	*20*	*41*	*86*	*>178*
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Caprio *et al.* (2015) derived a new global relationship by merging several databases collected in different geographical regions to highlight any systematic regional effects in the relationship between MMI and other ground motion parameters for the ShakeMap programs. They studied California, Italy, Greece, central eastern United States, and the global active crust. Within one standard deviation, their regional and global results are in agreement with the relations proposed by Worden *et al.* (2012) for California, Faenza and Michelini (2010) for Italy, Tselentis and Danciu (2008) for Greece, and Atkinson and Kaka (2007) for central eastern United States. The earthquakes in their study ranged in magnitude from 2.5 to 7.3, and the distances up to 200 km from the epicenter. They proposed the following two equations where coefficients are given in Table 2.5:

$$MMI = \alpha_1 + \beta_1 \log(PGM), \quad \text{if } \log(PGM) \leq t_{PGM} \tag{2.8}$$

$$MMI = \alpha_2 + \beta_2 \log(PGM), \quad \text{if } \log(PGM) > t_{PGM} \tag{2.9}$$

Table 2.5: Coefficients for relationship between ground motion and MMI (Caprio *et al.*, 2015)

Region	t_{PGM} [$\log(PGM)$]	α_1	β_1	σ_{x_1}	σ_{y_1}	α_2	β_2	σ_{x_2}	σ_{y_2}	t_{INT}
Global	PGV 0.3 $\pm$ 0.2	4.424	1.589	0.6	0.9	4.018	2.671	0.5	1.3	4.92$\pm$0.3
	PGA 1.6 $\pm$ 0.2	2.270	1.647	0.4	0.7	-1.361	3.822	0.4	1.4	4.87$\pm$0.3

Table 2.6 shows statistical information of calculated MMI values for all the 679 locations in Appendix C of NBC 2015. The MMI values are calculated using the relationships proposed by the aforementioned researchers: Wald et al. (1999), Atkinson and Kaka (2007), Worden et al. (2012), and Caprio et al. (2015). Worden et al. (2012) is an updated version of Wald et al. (1999) with almost the same researcher group, and therefore, it supersedes Wald et al. (1999), which has a limited database. Atkinson and Kaka (2007) includes central and eastern North America using the database from Wald et al. (1999). Caprio *et al.* (2015) has a larger database for California, Italy, Greece, Central-eastern United States, and the global active crust. Thus, in the following sections, the MMI values are based on Atkinson and Kaka (2007), Worden *et al.* (2012), and Caprio *et al.* (2015). Table 2.6 indicates that the difference between the MMI values based on these three studies is just one unit on the MMI scale. The results in Table 2.6 show that the considered models all predict similar MMI levels, with a minimum MMI between III and V (no real damage) over much

of central Canada, and peak MMI values of IX and X+ (hazardous damage to robust structures) for parts of Canada near the Cascadia subduction zone. The mean MMI calculated for all 679 locations in Appendix C of NBC 2015 corresponds to a damage level of no to light non-structural damage. The results shown in Table 2.6 indicate that, assuming *Site Class* C, a large portion of Canada is expected to experience no to light non-structural damage in the event of a design earthquake with a 2% probability of exceedance in 50 years.

Table 2.6: Statistical information for different MMI and PGV relationships

	PGV (m/s)	PGA (g)	W1999	KA2004-0	KA2004-1	KA2004-2	KA2004-3	AK2007	W2012	C2015
Minimum	0.020	0.030	3	4	4	4	3	5	4	4
Maximum	0.989	1.040	8	7	7	7	6	9	9	10
Median	0.079	0.085	5	5	5	5	4	6	5	6
Mean	0.147	0.153	5.2	5.2	5.3	4.8	4.6	6.1	5.6	6.2
Std. Dev.	0.178	0.153	1.4	0.8	0.8	0.7	0.7	1.0	1.3	1.2
W1999 (Wald *et al.*, 1999), KA2004-0 (Kaka and Atkinson, 2004, with no distance term); KA2004-1 (Kaka and Atkinson, 2004, with D=1 km); KA2004-2 (Kaka and Atkinson, 2004, with D=100 km); KA2004-3 (Kaka and Atkinson, 2004, with D=300 km); AK2007 (Atkinson and Kaka, 2007); W2012 (Worden *et al.*, 2012), and C2015 (Caprio *et al.*, 2015).										

It should be added that the the relationship proposed by Worden *et al.* (2012) is an updated version of that proposed by Wald *et al.* (1999). Worden et al. (2012) determined probabilistic relationships between ground-motion parameters and MMI in California based on approximately 200,000 MMI observations. The earthquakes in Worden *et al.* (1999) ranged in magnitude from 3.0 to 7.3, and the distances from the source ranged from less than a kilometer to approximately 400 km. For intensities of MMI $\geq$ 5, their results closely agreed with the relationships proposed by Wald *et al.* (1999). For lower intensities, their results fall midway between the relationships proposed by Wald *et al.* (1999) and those proposed by Atkinson and Kaka (2007).

The results in Table 2.6 indicate that the MMI for Eastern Canada calculated with the relationships proposed by Kaka and Atkinson (2004) for D = 1 km and D = 100 km are similar, but the MMI for D = 300 km are slightly lower.

The results also indicate that the MMI relationships by Wald *et al.* (1999), Atkinson and Kaka (2007), Worden *et al.* (2012), and Caprio *et al.* (2015) were self-consistent within one MMI unit over the entire range of Canadian seismicity considered, approximately 4.5 $\leq$ MMI $\leq$ 9.5. The MMI values calculated with the relationship by Wald *et al.* (1999) deviated significantly from the average for MMI < 5. Therefore, it was concluded that the relationships proposed by Atkinson and Kaka (2007), Worden *et al.* (2012), and Caprio *et al.* (2015) were reasonably consistent in predicting MMI over the entire range of Canadian seismicity. In this study, the empirical PGV-

MMI correlations proposed by Worden *et al.* (2012) were adopted to ensure consistency with practicing engineers familiar with the USGS ShakeMap program (Worden and Wald, 2016), and the given good agreement noted with other empirical relationships. There is no need to apply the differences in calculated MMI values for Eastern and Western Canada given that the NBC 2015 separates low seismicity areas (Eastern Canada and low seismic areas) from high seismic areas (mainly Western Canada), based on reference peak ground accelerations (PGAref).

It is also important to note that the relationship between MMI damage level and peak ground motion parameters are empirical, and are affected by factors such us density of records and building practices. Since the current study is consistent with the NBC 2015, the relationships developed by the authors can be revisited for consistency with subsequent revisions to the NBC, in order to check the possible effects of such changes on their applicability.

From the comparison with observed intensity maps, USGS (2005) finds that a regression based on peak velocity for intensity for MMI greater than VII and on peak acceleration for intensity less than VII is most suitable. This is consistent with the notion that low intensities are determined by felt accounts (sensitive to acceleration). Moderate damage, at intensity VI-VII, typically occurs in rigid structures (masonry walls, chimneys, etc.) that also are sensitive to high-frequency (acceleration) ground motions. As damage levels increase, damage also occurs in flexible structures in proportion to ground velocity, not acceleration. In the literature, PGV is more often cited as being a better indicator of potential damage than PGA (Sweny, 2012), and hence peak ground velocity was selected to delineate MMI levels in the present study.

The threshold PGV values demarcating different MMI damage levels adopted as the basis for defining the *site seismic categories* in the *Level 1 – PST* were proposed by NRCan (Halchuk, 2017). These thresholds, listed in Table 2.7, were mapped onto the MMI scale based on the work of Worden *et al.* (2012). For comparison, Table 2.7 also shows the PGV thresholds used to define seismic categorization as per NRCan (Halchuk, 2017) and Heidebrecht *et al.* (1983), and converted to MM. The NRCan (Halchuk, 2017) and Heidebrecht *et al.* (1983) PGV thresholds are similar. However, the NRCan thresholds are more convenient for practicing engineers since there are fewer regions to consider.

Table 2.7: *Building damage* levels correlated to PGV thresholds

***Building damage* level**	**MMI scale**	**Threshold PGV (m/s) mapped to MMI scale (Worden *et al.*, 2012)**			
		NRCan (Halchuk, 2017)		**Heidebrecht *et al.* (1983)**	
		>	≤	>	≤
No damage	V		0.050		0.05
Light non-structural damage	VI	0.050	0.100	0.05	0.10
Hazardous non-structural damage	VII	0.100	0.200	0.10	0.15
Light structural damage				0.15	0.20
Hazardous damage to susceptible structures	VIII	0.200	0.450	0.20	0.30
				0.30	0.40
Hazardous damage to robust structures	IX+	0.450		0.40	

2.1.3 Relationships between spectral response accelerations and peak ground velocity

The threshold values given in Table 2.7 correlate peak ground velocity – PGV – to damage level defined by the MMI scale. However, in seismic design, seismic hazard is commonly described through the use of a uniform hazard spectrum with respect to a specified probability of exceedance in 50 years; therefore it would be desirable to relate MMI damage level to code level spectral response accelerations. Using MMI-S_a thresholds has a number of advantages, including: (1) a better representation of *building* response, (2) familiarity of response spectrum to practicing engineers, and (3) the ability to readily incorporate the influence of soil conditions through existing site coefficients.

Seismic categorization is commonly conducted in terms of spectral acceleration parameters at short periods and long periods. For example, in ASCE 7, seismic design categories (SDCs) are tied to specific threshold values S_{DS} and S_{D1}, which are comparable to $F(0.2)S_a(0.2)$ and $F(1.0)S_a(1.0)$ in the NBC 2015. Although the NBC 2015 utilizes 5% damped spectral response accelerations at 0.2, 0.5, 1.0, 2.0, 5.0, and 10 seconds to cover the full spectral shape, it places restrictions on using different seismic force-resisting systems based on $I_EF(0.2)S_a(0.2)$ and $I_EF(1.0)S_a(1.0)$; respecting these indices would facilitate the applicability and practicality of any screening tool developed for the Canadian context.

As part of research collaboration with the Department of Earth Science at Carleton University, the MMI-PGV limits given in Table 2.7 were converted into MMI-S_a thresholds. This conversion was conducted by developing intermediate relationships between PGV and each of $S_a(0.2)$, $S_a(0.5)$, and $S_a(1.0)$, based on the Geological Survey of Canada's (GSC) "Fifth Generation Seismic Hazard Model Input Files" (Halchuk *et al.*, 2015) used to produce the NBC 2015 uniform hazard spectra with 2% probability of exceedance in 50 years, assuming *Site Class* C.

Different mathematical models have been tested, including:

$$Y = c_1 + c_2 PGV \tag{2.10}$$

$$Y = c_1 + c_2 PGV + c_3 PGV^2 \tag{2.11}$$

$$Y = c_1 + c_2 PGV + c_3 PGV^2 + c_4 PGV^3 \tag{2.12}$$

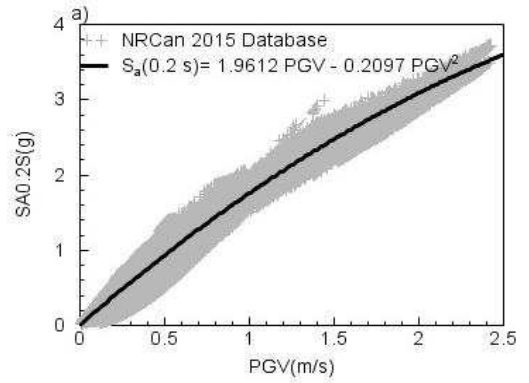
$$= c_1 + PGV^{c_2} \tag{2.13}$$

The quadratic Eq. (2.11) provides the best fit to the GSC database (Table 2.8), and therefore, it is used in the present study.

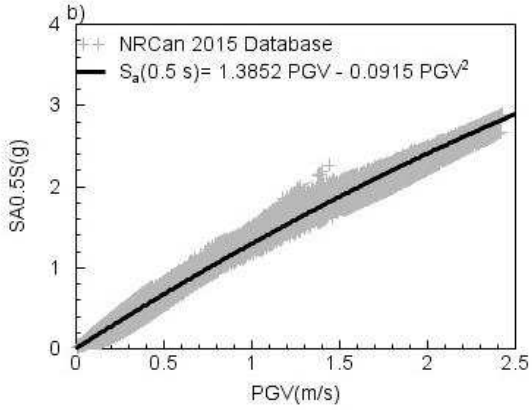
Table 2.8: S_a-PGV relationships based on the Geological Survey of Canada's (GSC) "Fifth Generation Seismic Hazard Model Input Files" databases (Halchuk *et al.*, 2015)

$Y=c_1+c_2\times PGV+c_3\times PGV^2$			
Y	$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$
c_1	0	0	0
c_2	1.9621	1.3852	0.8159
c_3	-0.2097	-0.0915	-0.0548
R^2	0.97	0.99	0.99

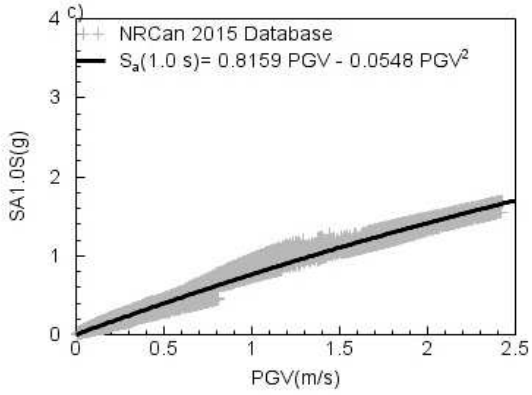
Figure 2.1 (a), (b) and (c) show the distribution of spectral response accelerations versus PGV for 0.2, 0.5 and 1.0 seconds, respectively, along with the curve fitting expressions of the form given in Table 2.8.



(a) $T = 0.2$ seconds



(b) $T = 0.5$ seconds



(c) $T = 1.0$ seconds

Figure 2.1: Relationship between PGV and $S_a(T)$ for T= 0.2, 0.5, and 1.0 seconds

2.1.4 Relationships between MMI scale and spectral response accelerations

The relationships developed for PGV-$S_a(T)$ shown in Figure 2.1 were mapped to the MMI-PGV bounds listed in Table 2.7. The results of the mapping process are shown in

Table 2.9.

The calculated mean MMI for each *site seismic category* is in good agreement with the established MMI damage levels. Furthermore, of the 679 locations listed in Appendix C of Division B of the NBC 2015, 153 locations are expected to experience ground shaking equal to MMI V generating no damage, and another 233 are expected to experience ground shaking equal to MMI VI generating light damage in non-structural elements. Similarly, the seismic hazard at 53 locations is consistent with MMI IX causing hazardous damage to robust structures.

Table 2.9: MMI-S_a thresholds based on PGV and S_a bounds

MMI scale	Halchuk (2017) Thresholds PGV (m/s)		Mean MMI	Max[S_a(0.2), S_a(0.5)]		S_a(1.0)		No. of locations† (679 total)
	>	≤		>	≤	>	≤	
MMI V		0.050	<5.8		0.098 g		0.041 g	153
MMI VI	0.050	0.100	6.2	0.098 g	0.194 g	0.041 g	0.081 g	233
MMI VII	0.100	0.200	7.0	0.194 g	0.384 g	0.081 g	0.161 g	131
MMI VIII	0.200	0.450	7.8	0.384 g	0.840 g	0.161 g	0.356 g	109
MMI IX+	0.450		>8.3	0.840 g		0.356 g		53

† – Calculated for *Site Class* C

2.1.5 Adjustment of spectral acceleration thresholds of site seismic categories

The MMI-S_a thresholds were found to show a striking similarity to the system restrictions placed on seismic force-resisting systems listed in Table 4.1.8.9 of the NBC 2015. Therefore, to maintain consistency with the NBC to the benefit of practising engineers, the MMI-S_a thresholds in

Table 2.9 were adjusted to align with NBC 2015 as follows:

1. A new region of Very Low *site seismic category* (SSC-0) was defined [corresponding to MMI V]. The threshold of the maximum of S_a(0.2) and S_a(0.5) was rounded to 0.1 g. The threshold of S_a(1.0) was back-calculated based on the PGV-$S_a(1.0)$ relationship. This region of Very Low *site seismic category* (SSC-0) was developed to reflect the need for *buildings* such as emergency facilities to remain operational and functional after a seismic event;

2. The threshold of the maximum of S_a(0.2) and S_a(0.5) for MMI VI, VII and VIII were slightly adjusted to 0.2 g, 0.35 g and 0.75 g, respectively, to suit the SFRS restrictions in the NBC 2015, with corresponding values of S_a(1.0) being back-calculated from the PGV-$S_a(1.0)$ relationships;

3. To reduce the ground motion variability and to provide a more uniform representation of regions of high seismicity in Canada, the MMI IX+ category was subdivided into two regions with intermediate bound applied to the maximum of S_a(0.2) and S_a(0.5) of 1.15 g and S_a(1.0) of 0.50 g; and

4. Site coefficients $F(T)$ are incorporated in the thresholds of *site seismic categories*.

It is noteworthy that the NBC 2015 only specifies one threshold of $S_a(1.0)$ for seismic design restrictions in Table 4.1.8.9. The adjusted thresholds for the six *site seismic categories* are shown in
Table 2.10, alongside the the approximate MMI scales.

Table 2.10: *Site seismic categories* and associated MMI scales

Site seismic category		***S*(0.2)**		***S*(1.0)**		**MMI Scale**
		>	≤	>	≤	
	Very Low (SSC-0)		0.10 g		0.05 g	MMI V
	Low (SSC-1)	0.10 g	0.20 g	0.05 g	0.10 g	MMI VI
	Moderate (SSC-2)	0.20 g	0.35 g	0.10 g	0.15 g	MMI VI½
	Moderately High (SSC-3)	0.35 g	0.75 g	0.15 g	0.30 g	MMI VII½
	High (SSC-4)	0.75 g	1.15 g	0.30 g	0.50 g	MMI VIII+
	Very High (SSC-5)	1.150 g		0.500 g		

The *site seismic category* based on $S(0.2)$ [maximum of $F(0.2)S_a(0.2)$ and $F(0.5)S_a(0.5)$] may be different from the *seismic category* based on $S(1.0)$ [$F(1.0)S_a(1.0)$]. In this case, select the higher as the *site seismic category*.

Figure 2.2 shows a *site seismic category* map assuming *Site Class* C. Based on this map, 406 out of 679 locations in Appendix C of the NBC 2015 fall into regions of Very Low and Low *site seismic categories* (SSC-0 and SSC-1), corresponding to anticipated damage levels of light non-structural damage or less. A small number of locations, predominately on the west coast, would be tagged as having Very High *site seismic category* (SSC-5), with very severe levels of damage expected. The map also shows that under typical soil condition (i.e., *Site Class* C), much of central Canada and portions of Eastern Canada would be exempt from the *Level 2 – SQST,* based on the threshold values defined by SSC-0 (no *damage*) and SSC-1 (light non-structural *damage*).

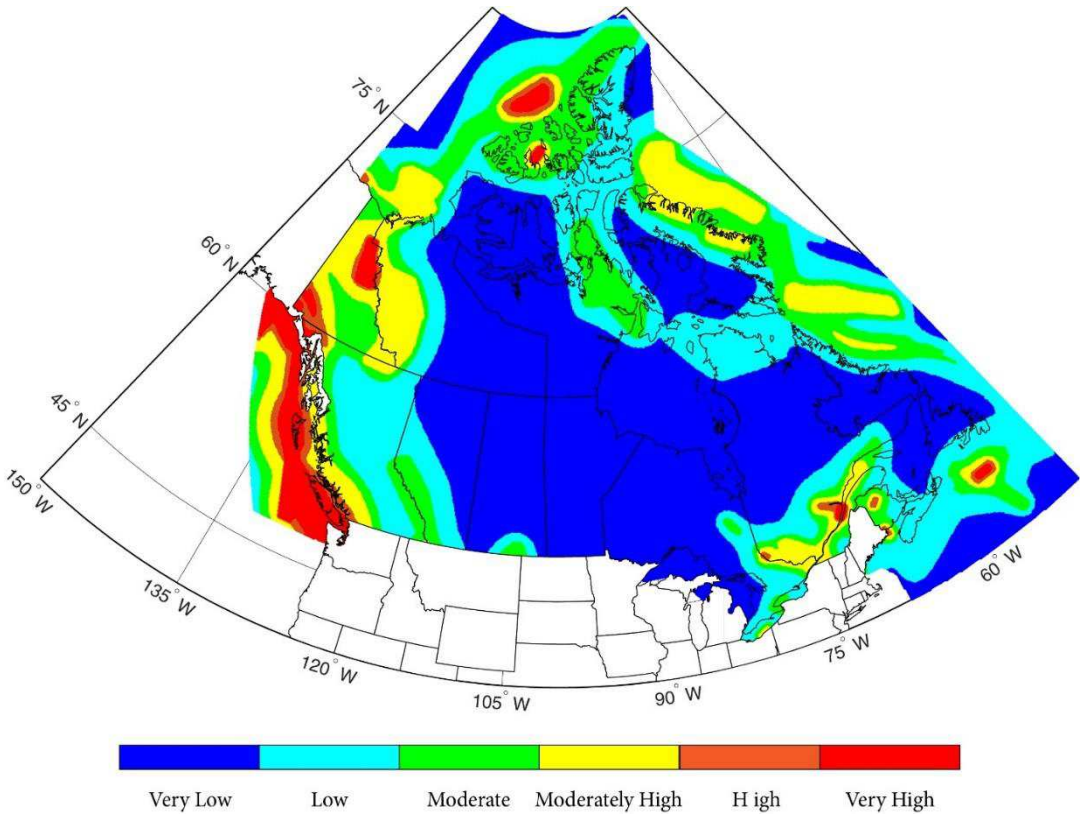


Figure 2.2: *Site seismic category* map for Canada assuming *Site Class* C

2.1.6 An example of determining site seismic category

Determination of the *site seismic category* of Alberni, British Columbia is presented as an example. The PGA, $S_a(0.2)$, $S_a(0.5)$, and $S_a(1.0)$ values for Alberni are 0.434 g, 0.955 g, 0.915 g and 0.594 g, respectively, for reference *Site Class* C, from Appendix C of Division B of the NBC 2015. Since $S_a(0.2)/\text{PGA} = 0.955/0.434 \geq 2$, $\text{PGA}_{\text{ref}} = 0.434$ g. In the NBC 2015, spectral response accelerations are adjusted for different *Site Classes* by multiplying $S_a(T)$ by site coefficients $F(T)$. $F(T)$ for *Site Classes* A, B, C, D, and E are provided in the NBC 2015. Note that the site coefficients for *Site Class* C are set to be unity.

If the location is in *Site Class* A, the procedure to determine the *site seismic category* is presented below:

1. Calculate $S(0.2) = S_a(0.2) \times F(0.2) = 0.955 \text{ g} \times 0.69 = 0.659$ g;
2. Calculate $S(0.5) = S_a(0.5) \times F(0.5) = 0.915 \text{ g} \times 0.57 = 0.522$ g;
3. Determine Max [$S(0.2)$, $S(0.5)$] = 0.659 g, which is in the range of SSC-3;
4. Calculate $S(1.0) = S_a(1.0) \times F(1.0) = 0.594 \text{ g} \times 0.57 = 0.339$ g, which is in the range of SSC-4.

The calculated Max [$S(0.2)$, $S(0.5)$] and $S(1.0)$ are in the ranges of SSC-3 and SSC-4, respectively. Thus, the site seismic category is the higher of SSC-3 and SSC-4, which is SSC-4.

If the same location is in *Site Class* C, then the site coefficients for Site Class C are unity because the site effect is already included in $S_a(0.2)$ and $S_a(1.0)$. The procedure to determine the site seismic category is presented below:

1. Calculate $S(0.2) = S_a(0.2) \times F(0.2) = 0.955$ g $\times$ 1.00 = 0.955 g;
2. Calculate $S(0.5) = S_a(0.5) \times F(0.5) = 0.915$ g $\times$ 1.00 = 0.915 g;
3. Determine Max [$S(0.2)$, $S(0.5)$] = 0.955 g, which is in the range of SSC-4;
4. *Calculate* $S(1.0) = S_a(1.0) \times F(1.0) = 0.594$ g $\times$ 1.00 = 0.594 g, which is in the range of SSC-5;

The calculated Max [$S(0.2)$, $S(0.5)$] and $S(1.0)$ are in the ranges of SSC-4 and SSC-5, respectively. Thus, the site seismic category is the higher of SSC-4 and SSC-5, which is SSC-5.

If the same location is in *Site Class* E, then:

1. Calculate $S(0.2) = S_a(0.2) \times F(0.2) = 0.955$ g $\times$ 0.90 = 0.860 g;
2. Calculate $S(0.5) = S_a(0.5) \times F(0.5) = 0.915$ g $\times$ 1.26 = 1.153 g;
3. Determine Max [$S(0.2)$, $S(0.5)$] = 1.153 g, which is in SSC-5;
4. Calculate $S(1.0) = S_a(1.0) \times F(1.0) = 0.594$ g $\times$ 1.53 = 0.909 g, which is in SSC-5.

The calculated Max [$S(0.2)$, $S(0.5)$] and $S(1.0)$ are both in the SSC-5 range. Thus, the *site seismic category* is SSC-5.

2.2 Benchmark NBC edition

2.2.1 Model building type

One of the key factors that influences a *building*'s seismic performance is the *model building type* defined in terms of the construction materials and type of seismic force-resisting system (SFRS). In the *Level 1 – PST*, *model building types* are largely based on the 1993 NRC screening manual, with two additional *model building types*, namely cold-formed steel (CFS) *buildings* and manufactured homes (MH). Including these two new *model building types* reflects the advances in seismic design/construction practices in the last two decades. Table 2.11 provides a summary of the names and descriptions used to define the *model building types*. The details of characteristics and typical damage of each *model building type* are provided in APPENDIX B.

Table 2.11: List of model building types in the *Level 1 – PST*

Model building type	Description	Material
WLF	Engineered Wood Light Frame *buildings* of up to 6 storeys in height, or having an area exceeding 600 m^2	Wood
WPB	Engineered Wood Post-and-Beam *buildings* which are covered by Part 4 of the NBC	
SMF	Steel Moment Frame	Steel
SBF	Steel Braced Frame	
SLF	Steel Light Frame	
SCW	Steel frame with Concrete shear Walls	Steel and concrete
SIW	Steel frame with Infill masonry shear Walls	Steel and masonry
CMF	Concrete Moment Frame	Concrete
CSW	Concrete Shear Walls	
CIW	Concrete frame with Infill masonry shear Walls	Concrete and masonry
PCW	Precast Concrete Walls	Precast concrete
PCF	Precast Concrete Frames	
RML	Reinforced Masonry bearing walls with Light wood or metal deck diaphragms	Masonry
RMC	Reinforced Masonry bearing walls with Concrete diaphragms	
URM	Un-Reinforced Masonry bearing wall buildings	
CFS	Cold-Formed Steel *buildings*	Other
MH	Manufactured Homes	

2.2.2 Identification of benchmark NBC editions

Generally, the NBC publishes periodic editions (i.e., every five years), which include revisions regarding seismic design requirements for new *buildings*. The NBC edition in which significantly improved seismic code requirements were adopted and enforced (which varies depending on *model building type*) is referred to as the *benchmark NBC edition*. Any building originally designed to or fully seismically upgraded to an applicable *benchmark NBC edition* or newer is referred to as a *post-benchmark building*.

The *benchmark NBC edition* varies depending on the *model building type* because the major improvements to the seismic provisions for different *model building types* may be adopted and enforced in different code editions. The identification of *benchmark NBC editions* is complex because it requires tracking the evolution of seismic design provisions in NBC editions and relevant standards to determine the specific code edition in which significantly improved seismic code provisions for a specific *building type* were adopted and enforced. Karbassi and Nollet (2008) identified the NBC 1970 as the *benchmark NBC edition* for all *building types*. The NBC 1970 was chosen because major changes were made in it. For example, the probabilistic seismic zoning maps

were first introduced, and the base shear was expressed as a function of building period. However, additional significant changes were made after the NBC 1970. For example, in the NBC 2005, uniform hazard spectra with a probability of 2% in 50 years were used for designing new buildings, and limitations and additional requirements for weak storey and other structural irregularities were introduced. Tischer (2014) adopted the NBC 1990 from the NRC screening manual (NRC, 1993a) as the *benchmark NBC edition* for all *building types* in an adapted seismic screening method. It is noteworthy that the NRC screening manual was qualitative based on seismic demand of *existing buildings*; the NBC 1990 was selected because the manual was made compatible with the NBC 1990. Saatcioglu et al. (2013) developed seismic screening software based on the NRC screening manual. The software is compatible with the seismic requirements set out in the NBC 2010, which was selected as the *benchmark NBC edition*. It is noteworthy that the studies by Tischer (2014) and Saatcioglu et al. (2013) did not investigate the major changes made in the seismic provisions in the NBC and relevant design standards for each specific *model building type*. Hence, the *benchmark NBC editions* in these studies could not be adopted in the *Level 1 – PST*.

In the United States, the *benchmark NBC editions* for different building types are determined by identifying the model building code editions in which significantly improved seismic requirements were adopted and enforced. Given the close technical cooperation that exists between Canadian and American engineers, many aspects of the NBC are essentially identical to those in the Uniform Building Code (UBC) (Uzumeri *et al.*, 1978). The evolution of the UBC and NBC is provided in APPENDIX C of this document. After determining the edition of the applicable UBC in which major seismic design improvements were adopted and enforced for a specified *building type*, the corresponding changes are tracked down in the NBC and relevant standards to determine the applicable *benchmark NBC edition* for the corresponding *model building type* in Canada. Table 2.12 presents the *benchmark NBC editions* and key detailed justifications for the seventeen *model building types*. More details on the determining the *benchmark NBC editions* can be found in APPENDIX A.

Table 2.12: *Benchmark NBC editions* and key justifications for seventeen model building types

Type	Benchmark NBC edition	Key justifications
WLF	2005 (≤ 4 *storeys*)	Seismic provisions for design of shear walls and horizontal diaphragms introduced in the 1989 edition of CSA O86 and provisions to mitigate the existence of the ground-floor soft storey and building irregularity introduced in the NBC 2005.
	2015 (> 4 *storeys* and ≤ 6 *storeys*)	Combustible construction up to 6 storeys permitted by NBC 2015.
WPB	2005	Seismic provisions for design of shear walls and horizontal diaphragms introduced in the 1989 edition of CSA O86 and provisions to mitigate the existence of the ground-floor soft storey and building irregularity introduced in the NBC 2005.
SMF	2005	Significant revisions to beam-column detailing in CSA S16-01, based on 1994 Northridge and 1995 Kobe earthquakes, were first referenced in NBC 2005.
SBF	2010 (buckling-restrained braced frames)	Buckling-restrained braced frames were first introduced in NBC 2010.
	2005 (other)	Major improvements in the design of steel-braced frames, including the connection seismic design, introduced into CSA S16-01 and referenced in NBC 2005.
SLF	2005	Similar performance to SMF *buildings* in transverse direction, and SBF *buildings* in longitudinal direction.
SCW	2005	Similar performance to CSW *buildings*.
SIW	2005	Similar performance to CIW *buildings*.
CMF	2015 (two-way slabs without beams)	Two-way slabs without beams were recognized as a type of CMF in the NBC 2015.
	2005 (other)	Significant improvement in seismic detailing of beam-column joints referenced in NBC 2005 via CSA A23.3-04. Different restrictions are introduced for different types of concrete SFRS, depending on their ductility level, seismicity, building importance, and soil type.
CSW	2005	The inherent overstrength of structural systems, as well as new restrictions on concrete SFRS and building irregularity, were introduced in NBC 2005.
CIW	2005	Significant revisions to masonry infill SFRS design introduced in CSA S304.1-04, and new restrictions on infill SFRS systems introduced in NBC 2005.
PCW	2015	Major improvements introduced in NBC 2015 via the CSA A23.3-14 for seismic design requirements for tilt-up *buildings*.
PCF	2005	Specific seismic design requirements for PCF SFRS introduced in CSA A23.3-04, and referenced in NBC 2005.
RML	2005	Improvements in diaphragm connection design referenced in the NBC 2005.
RMC	2005	NBC 2005 required torsional sensitivity analysis for *buildings* with rigid diaphragms and improvements in diaphragm connection design.
URM	2005	Stringent requirements imposed on URM SFRS systems beginning in NBC 2005.
CFS	2010	Seismic design provisions for *buildings* with cold-formed SFRS were first introduced in NBC 2010.
MH	2005 (< 4.3 m wide and 1 *storey*)	NBC 2005 explicitly required that small housing *buildings* to be anchored to their foundation in conformance with the requirements of CSA Z240.10.1-94.
	2010 (≥ 4.3 m wide or 2-3 *storeys*)	NBC 2010 imposed more stringent seismic anchorage requirements for MH *buildings* more than 4.3 m wide and with an overall height less than 3 storeys.

In the *Level 1 – PST*, *benchmark NBC editions* do not apply to non-structural components because non-structural components have been found to have been designed, installed, or modified without enforcement of applicable *building* code provisions (Masek and Ridge, 2009; ASCE, 2013).

The *benchmark NBC editions* in Table 2.12 correspond to the applicable NBC editions. It is typical that the provincial or municipal *building* code release dates in the areas screened lag behind the applicable NBC editions, and that the design and construction of a *building* lag behind the provincial or municipal *building* code release dates. In the seismic screening process, *benchmark NBC editions* shall only be identified based on well-documented sources, such as original structural drawings or by consulting with the local authority having jurisdiction.

2.3 Remaining occupancy time

An existing building with a shorter remaining occupancy life has a smaller chance of experiencing a code-level earthquake event over its remaining life (ASCE, 2017). Furthermore, for many publicly owned *buildings*, the time required for planning, conducting and completing a seismic retrofit may be greater than the remaining planned occupancy time of the building; hence a detailed evaluation of these structures is not warranted. A building is exempt from detailed *seismic risk* assessment if it can be shown that the *remaining occupancy time* is short enough to drop the seismic design spectra to an appreciably low level. This section describes the development of the *seismic risk* acceptance criterion based on the building's *remaining occupancy time*.

If a building is designed for an earthquake hazard with a constant probability of exceedance over its original design service life (i.e., 50 years), it becomes increasingly less likely that the building will experience the design earthquake hazard as it nears the end of its design service life. To demonstrate this concept, consider the expression for the probability of exceedance $P(n)$ in n years, based on the Poisson process (Commentary J of User's Guide – Part 4 of Division B of NBC 2010 Structural Commentaries), given below:

$$P(n) = 1 - e^{-P(\text{per year})\cdot n} \tag{2.14}$$

where $P(\text{per year}) = (1/\text{return period})$ denotes the probability of exceedance per year.

New *buildings* in Canada are designed for the effects of an earthquake with a probability of exceedance $P(n) = 2\%$ over a service life of 50 years. This yields an annual probability of exceedance $P(\text{per year}) = 0.0404\%$ and a return period of 2,475 years. Assuming a constant probability of exceedance of 2%, the corresponding $P(\text{per year})$ for an existing building with $n = 10$ years *remaining occupancy time* is 0. 202%, which is equal to a return period of 495 years. This is roughly equivalent to a 10% in 50 years seismic event, which corresponds to a return period of 475 years. Similarly, the return period for a building with 2 years of *remaining occupancy time* – for a hazard with a 2% chance of exceedance – is 97 years, which is equivalent to a 40% in 50 years earthquake with a return period of 98 years. Therefore, for a seismic hazard with a constant

probability of exceedance over a 50 year period, reducing the time horizon to less than the original service life of 50 years implies a reduction in the equivalent earthquake return period, and consequently, a reduction in the anticipated design response spectra.

In Australian Standard AS 1170.4 (2007), a probability factor k_p is introduced for the earthquake annual probability of exceedance (Table 2.13), which is applied to a baseline seismic hazard value with a return period of 500 years, in order to adjust the seismicity for different return periods between 20 and 2500 years. The use of a factor similar to k_p was of interest in developing the *Level 1 – PST* for *buildings* with a *remaining occupancy time* less than 50 years (by multiplying k_p to $F(T)S_a(T)$ thresholds for different *site seismic categories*). However, the applicability in Canada of the values for k_p proposed in Australian Standard AS 1170.4 (2007) needs further investigation.

Table 2.13: Annual Probability of Exceedance and Probability Factor k_p (AS1170.4, 2007)

Annual probability of exceedance, P	Probability factor k_p
1/2500	1.80
1/2000	1.70
1/1500	1.50
1/1000	1.30
1/800	1.25
1/500	1.00
1/250	0.75
1/200	0.70
1/100	0.50
1/50	0.35
1/25	0.25
1/20	0.20

Based on the preceding discussion, a *remaining occupancy time* factor, κ, was developed to account for the reduction in the probability of experiencing a Code-level earthquake for a building with a *remaining occupancy time* less than the design life of 50 years. The κ factor is used to scale response spectra to return periods other than 2 475 years following:

$$S(T)_{2\%/n} = \kappa \cdot S(T)_{2\%/50} \tag{2.15}$$

where $S(T)_{2\%/50}$ is the design spectral acceleration for a hazard with a probability of exceedance of 2% in 50 years, and $S(T)_{2\%/n}$ is the reduced design spectral acceleration for a hazard with a probability of exceedance of 2% in n years.

The *remaining occupancy time* reduction factor κ is a function of *remaining occupancy time* n and was developed by analyzing the variation in design spectral accelerations $S(T)$ at $T = 0.2$ and 1.0 seconds, for four different probabilities of exceedance values: 2% in 50 years, 5% in 50 years,

10% in 50 years, and 40% in 50 years. These data were obtained from Halchuk *et al.* (2015) for all 679 locations listed in NBC 2015. Using Eq. (2.15), these hazards with a given probability of exceedance such as 10% in 50 years may be converted into equivalent hazards with a 2% probability of exceedance in n years. Therefore, a hazard with a probability of exceedance of 5% in 50 years is equivalent to 2% in 20 years. Likewise, 10% in 50 years is equivalent to 2% in 10 years, and 40% in 50 years is equivalent to 2% in 2 years.

For each location and for each of the four annual probability of exceedance values considered, the design spectral accelerations for *Site Class* A through E are calculated for $T = 0.2$ and 1.0 seconds by multiplying $S_a(T)$ values by corresponding site coefficients. Then, for each location and for each *Site Class*, the κ factor is computed respectively for both $S(0.2)$ and $S(1.0)$ as the ratio of the design spectral accelerations for a hazard with 5% in 50 years, 10% in 50 years and 40% in 50 years, relative to a baseline hazard of 2% in 50 years. The κ factors obtained for both $T = 0.2$ and 1.0 seconds for all locations and *Site Classes* within each of the four probability of exceedance values were averaged to construct the relationship between κ and 2% in n years of *remaining occupancy time*, as shown in Figure 2.3. The coefficient of variations applied to κ were 7%, 13%, and 26% for an equivalent hazard with a probability of 2% in 20 years, 2% in 10 years, and 2% in 2 years, respectively. Based on the input hazard values for each of the four probabilities, κ was found to vary as a function of *remaining occupancy time* n between $\kappa = 1.00$ for n =50 years, and $\kappa = 0.17$ for n =2 years.

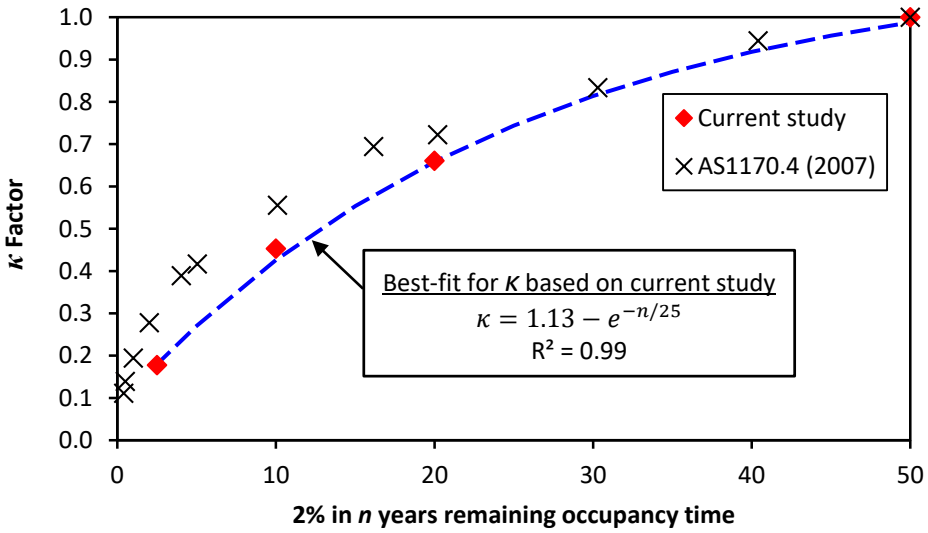


Figure 2.3: Relationship between κ and the hazard with 2% in n years remaining occupancy time

Figure 2.3 also shows a comparison of the AS1170.4 (2007) k_p factor, adjusted for a baseline corresponding to a return period of 2475 years, with the κ term of the current study. The

comparison shows that both seismicity adjustment factors yield similar estimates of reduced seismicity for different return periods. While both k_p and κ may be used to correlate seismic hazard values having different return periods, the κ factor was used to develop the *seismic risk* acceptance criteria based on *remaining occupancy time* in the *Level 1 – PST* since it is developed using Canadian earthquake records.

2.4 Consequences of failure

The failure of *buildings* can pose different levels of consequences associated with *seismic risk* to life safety. In the NBC 2015, low, normal, high, and post-disaster importance categories, which are defined based on use and *occupancy*, are used to address different *building'* consequences of failure. For example, schools are classified in the high importance category and therefore an importance factor of I_E = 1.3 is taken to increase the *building* capacity and in turn reduce the probability of failure with code-level earthquake shaking. However, the importance categories do not take into consideration other applicable key factors that contribute to the consequences of failure, such as *building* size, accessibility to escape, etc. For example, although a one-*storey* office *building* and a twenty-*storey* office *building* are classified in the same importance category (normal), the levels of consequences of failure for these two *buildings* differ significantly, given that the taller office building can accommodate far more *occupants* and requires more time to evacuate its *occupants*. In the *Level 1 – PST*, the consequence classification system proposed by Fathi-Fazl and Lounis (2017a) is adopted to consider existing buildings' consequences of failure. The system is based on the synthesis of different *building* consequences of failure classifications in existing codes, standards and guidelines in Canada and other countries. The proposed consequences of failure consist of three classes, namely: (i) Consequence Class – Low (CC-L), (ii) Consequence Class – Medium (CC-M), and (iii) Consequence Class - High (CC-H). These three classes are further divided into five levels, namely: Very Low Consequences, Low Consequences, Medium Consequences, High Consequences, and Very High Consequences. The key requirements that govern the consequence classification depend on the type of *occupancy*, *building* size, ability and mobility to escape, etc. *Consequences of failure* for different *building* occupancies are stipulated in Part 1: User's Guide of the *Level 1 – PST*.

2.5 Summary of seismic risk acceptance criteria

Table 2.14 presents a summary of the *seismic risk* acceptance criteria. The *remaining occupancy time* should not be applied to *buildings* of *Very High Consequences,* given the extremely high consequences in the event that these *buildings* fail.

Table 2.14: Seismic risk acceptance criteria

<table>
<tr><td rowspan="4">Site seismic category (SSC)</td><td colspan="3">Post-benchmark building</td><td colspan="4"><u>NOT</u> post-benchmark building</td></tr>
<tr><td>Structure</td><td colspan="2">Non-structural components</td><td colspan="4">Structure and non-structural components</td></tr>
<tr><td>COF</td><td colspan="2">Consequences of failure (COF)</td><td colspan="4">Consequences of failure (COF)</td></tr>
<tr><td>Any level</td><td>Any except VHC</td><td>VHC</td><td>VLC</td><td>LC & MC</td><td>HC</td><td>VHC</td></tr>
<tr><td>SSC-0</td><td rowspan="5">Met</td><td rowspan="2">Met</td><td>Met</td><td colspan="4">Met</td></tr>
<tr><td>SSC-1</td><td rowspan="4">Not Met</td><td colspan="2" rowspan="2">Met</td><td>Met if $n \leq 10$;
Not Met if $n > 10$</td><td rowspan="4">Not Met</td></tr>
<tr><td>SSC-2</td><td rowspan="3">Not Met</td><td colspan="2">Met if $n \leq 10$; Not Met if $n > 10$</td><td>Met if $n \leq 5$;
Not Met if $n > 5$</td></tr>
<tr><td>SSC-3</td><td colspan="2">Met if $n \leq 5$; Not Met if $n > 5$</td><td colspan="2" rowspan="2">Not Met</td></tr>
<tr><td>SSC-4 & SSC-5</td><td colspan="2">Not Met</td></tr>
</table>

Note: n refers to the remaining occupancy time (years).

2.6 Building deterioration and damage

Building deterioration and *damage* have the potential to impact an *existing building's* expected performance. Therefore, it is critical to consider this matter during *seismic risk* assessment. In the presence of any *deterioration* or *damage* (e.g., extensive dry rot, deformation, corrosion, or concrete spalling), the *building* is required to be further assessed using the *Level 2 – SQST*. In other words, the *Level 1 – PST* can only be used in cases where there is no history of *deterioration* or *damage* that has not been repaired at the time of screening.

2.7 Special conditions triggering the Level 3 – SEG

2.7.1 Unknown model building type

An *existing building's* seismic performance highly depends on its *model building type,* which is defined according to its construction materials and type of seismic force-resisting system (SFRS). If the *model building type* is unknown, the *building's* seismic resistance is largely uncertain. Consequently, the *Level 3 – SEG* should be triggered to further investigate the *building's seismic risk.*

2.7.2 Federal heritage designation

A *building* identified as *federal heritage designation* is one that has a profound importance to Canada by virtue of its direct association with a nationally significant aspect of Canadian history (Historic Sites and Monuments Board of Canada 2008). Given the potential social consequences resulting from the failure of such highly regarded *buildings*, *buildings* identified as *federal heritage designations* are immediately be flagged for *Level 3 – SEG*.

2.7.3 Change of occupancy increasing structural loads

An existing *building's* occupancy may change for different reasons during its service life. This change may increase structural loads other than seismic. If this is the case, the *Level 3 – SEG* should be triggered to assess the effect that the change in structural loads has on the *building's* seismic performance.

2.7.4 Consequences of failure higher than original consequences of failure

The change in *building occupancy* may result in increased consequences of failure. Consequences of failure may also increase as a result of *building* addition(s) that affects the *number of storeys* and *total floor area*. The *Level 1 – PST* does not address the effect of increased consequences of failure. *Level 3 – SEG* should be triggered to assess the effect of increased consequences of failure in existing buildings.

2.7.5 Site Class F

According to the current edition of the NBC, no site coefficients are provided for *Site Class* F (such as liquefiable soils and quick and highly sensitive clays), and site-specific evaluation is required. Since *Site Class* F's effect on buildings' seismic performance is highly uncertain, the *Level 1 – PST* flags *buildings* with *Site Class* F for the *Level 3 – SEG*.

2.7.6 Geologic hazards

Geologic hazards are conditions present at or near the *building* foundation with the potential to significantly increase the *building*'s seismic vulnerability. Given the potential negative effect, *buildings* located on or near any *geologic hazards* should be immediately flagged for the *Level 3 – SEG*. In the *Level 1 – PST*, *geologic hazards* include liquefaction, landslide potential, and surface fault rupture.

3.0 EVALUATION OF SEISMIC RISK ACCEPTANCE CRITERIA

3.1 General

The *Level 1 – PST* provides *seismic risk* acceptance criteria that can be used to exempt *existing buildings* from the *Level 2 – SQST*. This section presents the evaluation of the *seismic risk* acceptance criteria using the *Level 2 – SQST* and HAZUS criteria.

3.2 Overview of the Level 2 – SQST

The *Level 2 – SQST* developed by Fathi-Fazl *et al.* (2018) is intended to be the second level of a *seismic risk* management framework proposed by Lounis *et al.* (2016). It is preceded by the *Level 1 – PST*. The *Level 2 – SQST* consists of a quantitative structural *seismic risk* scoring system and a qualitative *seismic risk* scoring system for non-structural components. Structural scores and non-structural component scores are calculated separately and compared with the corresponding structural and non-structural component thresholds to determine whether a *building's seismic risk* is acceptable.

In the *Level 2 – SQST*, structural score, S, is defined as the negative common logarithm of the probability of collapse given a code level earthquake (CLE) associated with a specified *seismic zone*, i.e.,

$$S = -\log_{10}[P(COL|CLE)] \tag{3.1}$$

Note that the *seismic zones* in the *Level 2 – SQST* correspond to the *site seismic categories* for *Site Class* C in the *Level 1 – PST*.

The structural score S can be also expressed as:

$$S = S_B + \sum_i M_i \tag{3.2}$$

where: S_B is a basic structural score that is related to the probability of collapse for a *given model building type* and *seismic zone*, and M_i is the structural score modifier accounting for the effect of i^{th} parameter on the *building's* seismic performance.

The structural score S is compared with a corresponding structural score threshold S_{TH} in Table 3.1 to determine whether the structural *seismic risk* is acceptable (Fathi-Fazl *et al.*, 2018).

Table 3.1. Structural score thresholds and corresponding probability of collapse in 50 years

Consequence of failure	Structural score thresholds, S_{TH}	Probability of collapse in 50 years
Very Low	1.7	2%
Low & Medium	2.0	1%
High	2.3	0.5%
Very High	2.6	0.25%

3.3 Evaluation of seismic risk acceptance criteria based on seismicity

3.3.1 Structural seismic risk

The minimum structural score ($S_{\min}$) is associated with an *existing building* for which all deficiencies are present simultaneously. Since the *Level 1 – PST* assumes that the *building* is in good condition, the *building deterioration/damage* is not considered in calculating $S_{\min}$. $S_{\min}$ values for all *model building types* in Very Low and Low *site seismic categories* (SSC-0 and SSC-1) were calculated using the *Level 2 – SQST*. The results are presented in Table 3.2.

Table 3.2. Minimum structural scores for *existing buildings* located in Very Low and Low *site seismic categories*

Site seismic category	WLF	WPB	SMF	SBF	SLF	SCW	SIW	CMF	CSW	CIW	PCF	PCW	RML	RMC	URM	MH
Very Low (SSC-0)	5.4	5.4	5.4	3.3	3.5	3.7	3.9	3.8	2.9	4.0	3.4	3.6	3.3	3.8	3.8	3.3
Low (SSC-1)	4.1	4.0	4.0	2.3	2.5	2.7	2.7	2.6	1.9	2.7	2.2	2.5	2.1	2.5	2.5	2.1

The results in Table 3.2 show that the $S_{\min}$ values for all *model building types* in the Very Low *site seismic category* (SSC-0) are well above structural thresholds S_{TH} corresponding to all levels of consequences of failure (Table 3.1). Therefore, the structural *seismic risk* of *existing buildings* in this *site seismic category* is acceptable. In the Low *site seismic category* (SSC-1), $S_{\min}$ values are greater than or roughly equal to S_{TH} values for *Very Low*, *Low*, and *Medium Consequences*. Therefore, the structural *seismic risk* of *existing buildings* with *Very Low*, *Low*, and *Medium Consequences* is acceptable if located in Very Low and Low *site seismic categories*.

3.3.2 Seismic risk of non-structural components

The *seismic risk* acceptance criteria for non-structural components in Very Low and Low *site seismic categories* are evaluated by comparing the expected drift ratios and spectral accelerations for all *model building types* to HAZUS median drift ratio and peak floor acceleration limits. The median drift ratio Δ_d proposed by HAZUS (FEMA, 2003) for slight damage to drift-sensitive non-structural components is 0.4%, while the median peak floor acceleration A_{max} that triggers the slight damage to acceleration-sensitive non-structural components is 0.2 g. The Δ_d and A_{max}

limits are assumed to be appropriate for special *buildings* (equivalent to the *buildings* with *Very High Consequences* in the *Level 1 – PST*).

It should be recalled that the upper bound of spectral response acceleration at short periods for the Very Low and Low *site seismic categories* are 0.1 g and 0.2 g, respectively. Considering higher mode effects and/or building irregularity, acceleration-sensitive non-structural components may experience slight damage ($A_{max} > 0.2\ g$) in the Low *site seismic category* (SSC-1) but are unlikely to experience any *damage* in the Very Low *site seismic category* (SSC-0).

To assess *existing building's* drift ratios, a simplified manner is adopted using response spectrum analysis for acceleration- and velocity-controlled response (Chopra, 2007):

$$u_{max} = \begin{cases} \dfrac{T^2}{\alpha_2} \cdot \dfrac{\mathrm{Max}[S_a(0.2), S_a(0.5)]}{4\pi^2} & \text{for } T < T_s \\ \dfrac{T}{\alpha_2} \cdot \dfrac{S_a(1.0)}{4\pi^2} & \text{for } T \geq T_s \end{cases} \tag{3.3}$$

where: α_2 is the modal height factor used to convert the *building* to an equivalent single degree-of-freedom system, and T_S is the transition period between acceleration- and velocity-controlled response given by

$$T_S = \left\{ \frac{\mathrm{Max}[S_a(0.2), S_a(0.5)]}{S_a(1.0)} \right\} \tag{3.4}$$

The drift ratio is calculated for five generalized *building types* as per the NBC 2015 in low-, mid- and high-rise *building* height configurations based on a typical storey height of 4 m. To be consistent with the *building* height configuration in the *Level 2 – SQST*, the low-rise configuration was assumed to be 3 storeys in height ($H = 12\ m$), while the mid- and high-rises are 7 storeys ($H = 28\ m$) and 15 storeys ($H = 60\ m$), respectively. The natural periods T for the five generalized *building types* are computed based on Article 4.1.8.11 of the NBC 2015, representing the typical range of stiffness and mass for each type of SFRS.

$T = 0.085(H)^{3/4}$	For steel moment frames (Type 1)
$T = 0.075(H)^{3/4}$	For concrete moment frames (Type 2)
$T = 0.1\mathrm{n_S}$	For other moment frames (Type 3), $\mathrm{n_s}$ = number of storeys
$T = 0.025H$	For braced frames (Type 4)
$T = 0.05(H)^{3/4}$	For shear walls and other structures (Type 5)

In the calculations, the following α_2 are used: 0.75 for low-rise *buildings*, 0.69 for mid-rise *buildings*, and 0.60 for high-rise *buildings*. Values of spectral accelerations used to calculate drift ratios are taken from

Table 2.10 for the Very Low through Very High *site seismic categories* for *Site Class* C.

The calculated drift ratios are provided in Table 3.3. The maximum drift ratio for all *building types* was calculated to be 0.04% and 0.11% for the Very Low and Low *site seismic categories* (SSC-0 and SSC-1), respectively.

Table 3.3: Maximum drift ratios of all five generalized *building types*

Site seismic categories	Height	d/H (%)				
		Type 1	Type 2	Type 3	Type 4	Type 5
Very Low	low-rise	0.04	0.04	0.01	0.01	0.02
	mid-rise	0.04	0.04	0.03	0.03	0.02
	high-rise	0.04	0.04	0.03	0.03	0.02
Low	low-rise	0.11	0.09	0.04	0.04	0.04
	mid-rise	0.09	0.08	0.06	0.06	0.06
	high-rise	0.09	0.08	0.07	0.07	0.05

Even though the threshold of $\Delta_d = 0.4\%$ triggering slight damage is reduced by a factor of 2 for plan irregularity and further reduced by a factor of 1.25 for severe vertical irregularity (FEMA, 2015), the maximum drift ratio as shown in Table 3.3 is still less than the reduced drift threshold. Therefore, drift-sensitive non-structural components are unlikely to experience *damage* in the Very Low and Low *site seismic categories*.

3.4 Evaluation of seismic risk acceptance criteria based on remaining occupancy time

The *Level 1 – PST* provides *seismic risk* acceptance criteria based on the *remaining occupancy time* of the *building*. A *remaining occupancy time* factor κ is used to reduce the code level seismic hazard for an *existing building* with *remaining occupancy time* less than 50 years. In the *Level 1 – PST*, *remaining occupancy times* $n \leq 5$ and $n \leq 10$ are considered, resulting in κ values of 0.28 and 0.44, respectively. When the 0.28 value is applied to the thresholds in Table 2.10, SSC-3 drops to SSC-1, and when the 0.44 value is applied to the thresholds in Table 2.10, SSC-2 drops to SSC-1. Since the *seismic risk* acceptance criteria based on *seismicity* has been evaluated, the *seismic risk* of *buildings* located in SSC-2 and SSC-3 is acceptable if their *remaining occupancy times* are not greater than 10 years and 5 years, respectively.

4.0 REFERENCES

ACI 318. 1971. Building code requirements for reinforced concrete. American Concrete Institute, MI, USA.

Adebar, P., Mutrie, J.G., DeVall, R., and Mitchell, D. 2014. Seismic design of concrete buildings: the 2015 Canadian Building Code. Tenth U.S. National Conference on Earthquake Engineering Frontiers of Earthquake Engineering July 21-25, Anchorage, Alaska.

AISC. 1997. Seismic provisions for structural steel buildings. American Institute of Steel Construction, Chicago, IL.

Allen, T.I., Adams, J., and Halchuk, S. 2015a. The seismic hazard model for Canada: past, present and future. Proceedings of the Tenth Pacific Conference on Earthquake Engineering: Building an earthquake-resilient pacific, 6-8 November, Sydney, Australia.

Allen, T.I., Luco, N., and Halchuk, S. 2015b. Exploring risk-targeted ground motions for the National Building Code of Canada. 11th Canadian Conference on Earthquake Engineering, Victoria, Canada, Paper 93781.

Anderson, D., Brzev, S. 2009. Seismic design guide for masonry buildings. Canadian Concrete Masonry Producers Association.

AS 1170-4. 2007. Structural design actions – Part 4: earthquake actions in Australia.

AS-NZS 1170-2. 2002. Structural design actions – Part 0: General principles. Authority of New Zealand Structure Verification Method B1/VM1, NZ.

ASCE 31. 2003. Seismic evaluation of existing buildings. American Society of Civil Engineers, Reston, VA.

ASCE 41. 2006. Seismic rehabilitation of existing buildings. American Society of Civil Engineers, Reston, VA.

ASCE 41. 2014. Seismic rehabilitation of existing buildings. ASCE/SEI Standard 41-13, American Society of Civil Engineers, Reston, VA.

ASCE 7. 2010. Minimum design loads for buildings and other structures. ISBN: 9780784410851, American Society of Civil Engineers, Washington D.C.

ATC. 1978. Tentative provisions for the development of seismic regulations for buildings (ATC-3), prepared by the Applied Technology Council. Associated with the Structural Engineers Association of California Washington D.C.: National Bureau of Standards.

Atkinson, G.M. and Kaka S.I. 2007. Relationships between felt intensity and instrumental ground motion in the central United States and California. B. Seismol. Soc. Am., p. 497-510.

Atkinson, G.M. and Morrison, M. 2009. Observations on regional variability in ground-motion amplitudes for small-to-moderate earthquakes in North America. B. Seismol. Soc. Am., 99 (4): p. 2393-2409.

Basham, P.W., Weichert, D.H., Anglin, F.M., and Berry, M.J. 1985. New probabilistic strong seismic ground motion maps of Canada. Bulletin of the Seismological Society of America, 75: 563–595.

British Constructional Steelwork Association, Ltd. 2017. Portal frame building image. Retrieved April 10, 2017, from http://www.steelconstruction.info/Single_storey_industrial_buildings.

Bruneau, M. 1995. Performance of masonry structures during the 1994 Northridge (Los Angeles) earthquake. Canadian Journal of Civil Engineering, 22: 378-402.

Buchanan, A. H. 1983. Developments in design of wood structures for earthquake resistance. Fourth Canadian Conference on Earthquake Engineering. Vancouver, BC.

Bussell, M. 1997. Appraisal of existing iron and steel structures. The Steel Construction Institute, Ascot, England, SCI Publication 138.

CAC. 2003. Cement Association of Canada. Tilt-up concrete wall panels – design standard guide. https://www.scribd.com/document/217585651/Canada-Standards-Guide-for-Tilt-Up.

CAC. 2010. California administrative code, seismic evaluation procedures for hospital buildings. 2010, California Code of Regulations, Title 24, Part 1, Chapter 6.

Caprio M., Tarigan B., Worden C. B., Wiemer S., Wald D. J. 2015. Ground motion to intensity conversion equations (GMICEs): a global relationship and evaluation of regional dependency. Bulletin of the Seismological Society of America, p. 1476–1490.

Chandramohan, R., Baker, J.W., and Deierlein, G.G. 2015. Quantifying the influence of ground motion duration on structural collapse capacity using spectrally equivalent records. Earthquake Spectra, doi:10.1193/122813EQS298MR2.

CFSEI (Cold-Formed Steel Engineers Institute). 2017. History. Retrieved on February 2008, from http://www.cfsei.org/history.

CPCI. 1987. Design manual. Canadian Precast Prestressed Concrete Institute. Ottawa, ON.

CSA A23.3. 1959. Reinforced concrete design. Canadian Standards Association, Rexdale, Ont.

CSA A23.3. 1973. Code for the design of concrete structures for buildings. Canadian Standards Association, Rexdale, ON.

CSA A23.3. 1977. Code for the design of concrete structures for buildings. Canadian Standards Association, Rexdale, ON.

CSA A23.3. 1984. Design of concrete structures for buildings. Canadian Standards Association, Rexdale, ON.

CSA A23.3. 1994. Design of concrete structures. Canadian Standards Association, Rexdale, ON.

CSA A23.3. 2004. Design of concrete structures. Canadian Standards Association, Rexdale, ON.

CSA A23.3. 2014. Design of concrete structures. Canadian Standards Association, Rexdale, ON.

CSA O86. 2001. Engineering design in wood. Canadian Standards Association, Rexdale, ON.

CSA O86. 2009. Engineering design in wood. Canadian Standards Association, Rexdale, ON.

CSA O86. 2014. Engineering design in wood. Canadian Standards Association, Rexdale, ON.

CSA S136. 2007. North American specification for the design of cold-formed steel structural members. Canadian Standards Association, Rexdale, ON.

CSA S136. 2012. North American specification for the design of cold-formed steel structural members. Canadian Standards Association, Rexdale, ON.

CSA S136. 2016. North American specification for the design of cold-formed steel structural members. Canadian Standards Association, Rexdale, ON.

CSA S16. 2001. Design of steel structures. Canadian Standards Association, Rexdale, ON.

CSA S16. 2014. Design of steel structures. Canadian Standards Association, Rexdale, ON.

CSA S304.1. 1994. Masonry design for buildings (limit states design). Canadian Standards Association, Rexdale, ON.

CSA S304.1. 2004. Design of masonry structures. Canadian Standards Association, Rexdale, ON.

CSA S304.1. 2014. Design of masonry structures. Canadian Standards Association, Rexdale, ON.

CSA S832. 2006. Seismic risk reduction of operational and functional components (OFCs) of buildings. Canadian Standards Association, Rexdale, ON.

CSA S832. 2014. Seismic risk reduction of operational and functional components (OFCs) of buildings. Canadian Standards Association, Rexdale, ON.

CSA Z240.10.1. 1994. Site preparation, foundation and anchorage of mobile homes. Canadian Standards Association, Rexdale, ON.

CSA Z240.10.1. 2016. Site preparation, foundation and installation of buildings. Canadian Standards Association, Rexdale, ON.

CWC. 2003. Canadian Wood Council. Wood-frame construction: meeting the challenges of earthquakes. Building Performance Series No. 5. Ottawa. ON.

DeVall, R.H. 2003. Background information for some of the proposed earthquake design provisions for the 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, 30: 279–286.

Devine, F., Olund, O., Elwood, K., and Adebar, P. 2008. Seismic performance of concrete tilt-up buildings: current wall-to-slab connections. The 14th World Conference on Earthquake Engineering, October 12-17, Beijing, China.

EERI. 1996. Northridge reconnaissance report. Earthquake Spectra, Earthquake Engineering Research Institute, Oakland, CA, Supplement C to Vol. 11, pp. 25-48.

Faenza L. and Michelini A. 2010. Regression analysis of MCS intensity and ground motion parameters in Italy and its application in ShakeMap. Geophys. J. Int, p. 1138–1152.

Fathi-Fazl, R., Lounis, Z., (2017). Identification of consequences of failure criteria for seismic risk management of existing buildings. National Research Council of Canada, Ottawa, Ont. NRC Report No. A1-011750.

Fathi-Fazl, R., Jacques, E., Kadhom, B., Saassouh, B., Cai, Z., Cortes, L., Singh, J. (2017). Semi-quantitative seismic risk screening tool for existing buildings (Volume II), in progress. National Research Council of Canada, Ottawa, Ont. NRC Report No. A1-010575.

FEMA 154. 1988. Rapid visual screening of buildings for potential seismic hazards: a handbook. U.S. Federal Emergency Management Agency, First Edition, Washington, D.C.

FEMA 154. 2002. Rapid visual screening of buildings for potential seismic hazards: a handbook. U.S. Federal Emergency Management Agency, Second Edition, Washington, D.C.

FEMA 155. 1988. Rapid visual screening of buildings for potential seismic hazards: supporting documentation. U.S. Federal Emergency Management Agency, First Edition, Washington, D.C.

FEMA 155. 2002. Rapid visual screening of buildings for potential seismic hazards: supporting documentation. U.S. Federal Emergency Management Agency, Second Edition, Washington, D.C.

FEMA 178. 1992. NEHRP handbook of techniques for the seismic rehabilitation of existing buildings. U.S. Federal Emergency Management Agency, Washington, D.C.

FEMA 1985. NEHRP recommended provisions for seismic regulations for new buildings, Parts 1 and 2. Federal Emergency Management Agency, Building Seismic Safety Council, Washington, DC, February 1986.

FEMA 2003. Multi-hazard loss estimation methodology, Earthquake model, HAZUS-MH MR1, Advanced engineering building module, Technical and user's manual, developed by Department of Homeland Security, Emergency Preparedness and Response Directorate, Mitigation Division under a contract with National Institute of Buildings Sciences. Washington, D.C.

FEMA 303. 1997. NEHRP recommended provisions for seismic regulations for new buildings and other structures, Part 2: commentary. National Earthquake Hazard Reduction Program. Building Seismic Safety Council, Washington, D.C.

FEMA 310. 1998. Handbook for seismic evaluation of buildings – a pre-standard. U.S. Federal Emergency Management Agency, Washington, D.C.

FEMA 356. 2000. Pre-standard and commentary for the seismic rehabilitation of buildings. U.S. Federal Emergency Management Agency, Washington, D.C.

FEMA 547. 2006. Techniques for the seismic rehabilitation of existing buildings. U.S. Federal Emergency Management Agency, Washington, D.C.

FEMA 774. 2009. Unreinforced masonry buildings and earthquakes – developing successful risk reduction programs. U.S. Federal Emergency Management Agency, Washington, D.C.

FEMA E-74. 2012. Reducing the risks of nonstructural earthquake damage – a practical guide. U.S. Federal Emergency Management Agency, First Edition, Washington, D.C.

FEMA P-154. 2015. Rapid visual screening of buildings for potential seismic hazards: a handbook. U.S. Federal Emergency Management Agency, Third Edition, Washington, D.C.

FEMA P-155 2015. Rapid visual screening of buildings for potential seismic hazards: supporting documentation. U.S. Federal Emergency Management Agency, Third Edition, Washington, D.C.

Ghosh, S. K. 2001a. Seismic design provisions for precast concrete structures in ACI 318. PCI Journal, 46: 1, Publisher: Precast/Prestressed Concrete Institute ISSN: 0887-9672.

Ghosh, S. K. 2001b. Comparison of the seismic provisions of the 1997 Uniform Building Code to the 1997 NEHRP recommended provisions. A report to: Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, MD, 20899.

Ghosh, S. K. 2002. Building codes, standards and resource documents: a status report. PCI Journal, 47(2), March-April.

Halchuk S., Allen T. I., Adams J., Rogers G. C. 2015. Fifth generation seismic hazard model input files as proposed to produce values for the 2015 National Building Code of Canada. Natural Resources Canada, Open file 7893, Geological Survey of Canada, Ottawa, ON.

Heidebrecht, A.C., Basham, P.W., Rainer, J.H., and Berry, M.J. 1983. Engineering applications of new seismic ground-motion maps of Canada. Canadian Journal of Civil Engineering, 10(2): 670–680.

Heidebrecht, A.C. 2003. Overview of seismic provisions of the proposed 2005 edition of National Building Code of Canada. Canadian Journal of Civil Engineering, 30(2): 241–254. doi:10.1139/l02-068.

Housner, G.W. 1971. General features of the San Fernando earthquake, in Engineering Features of the San Fernando Earthquake, February 9, 1971. Edited by P. C. Jennings. Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, Calif., Report No. EERL 71-02.

Humar, J. 2015. Background to some of the seismic design provisions of the 2015 National Building Code of Canada. Canadian Journal of Civil Engineering, Volume 42, Number 11, pp. 940-952(13).

IBC. 2000. International Building Code. International Code Council.

IBC. 2006. International Building Code. International Code Council.

IRC. 2006. International Residential Code. International Code Council.

Jain, S.K, Mira, K., Kumar, M. and Shah, M. 2010. A proposed rapid visual screening procedure for seismic evaluation of RC-frame buildings in India. Earthquake Spectra, Volume 26, No. 3, pages 709-729, August.

Kaka and Atkinson. 2004. Relationships between instrumental ground-motion parameters and Modified Mercalli Intensity in eastern North America. Bulletin of the Seismological Society of America, Vol. 94, No. 5, pp. 1728–1736.

Karmod Prefabricated Technologies. 2017. Prefabricated building image. Retrieved March 30, 2017, from http://blog.karmod.eu/prefabricated-buildings/.

Klingner, R.E. 2004. Behavior of masonry in the Northridge (US) and Tecomán-Colima (Mexico) earthquakes: lessons learned, and changes in US design provisions, construction and building materials. 20(4), 209-219. doi: 10.1016/j.conbuildmat.2005.08.024.

Liel, A.B., Luco, N., Raghunandan, M. and Champion, C.P. 2015. Modifications to risk-targeted seismic design maps for subduction and near-fault hazards. 12th International Conference on Applications of Statistics and Probability in Civil Engineering, Vancouver, Canada.

Lounis, Z. Fathi-Fazl, R., Almansour, H., Cusson, D., and Singh, J. 2016. Framework for seismic risk management of existing buildings. National Research Council of Canada, Ottawa, Ont. NRC Report No. A1-008205.

Luco, N., Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K. and Kircher, C.A., 2007. Risk-targeted versus current seismic design maps for the conterminous United States. Structural Engineers Association of California, Convention Proceedings, pp. 163-175.

Masek, M. and Ridge, R. 2009. Identification of methods to achieve successful implementation of nonstructural and equipment seismic restraints. Earthquake Engineering Research Institute, Oakland, CA.

Mitchell, D., and Cook, W.D. 1984. Preventing progressive collapse of slab structures. ASCE Journal of Structural Engineering, 110 (7): 15 13 – 1532.

Mitchell, D., DeVall, R.H., Saatcioglu, M., Simpson, R., Tinawi, R., and Tremblay, R. 1995. Damage to concrete structures due to the 1994 Northridge earthquake. Canadian Journal of Civil Engineering. 22: 361-377.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., and L. Anderson, D. 2003. Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering. 30: 308–327.

Moehle, J. P., Hooper, J. D., Kelly, D. J., and Meyer, T.R. 2011. Seismic design of cast-in-place concrete diaphragms, chords, and collectors: a guide for practicing engineers. NEHRP Seismic Design Technical Brief No. 3, NIST GCR 10-917-4

Morgan, S. 2012. Retrieved March 30, 2017, from https://en.wikipedia.org/wiki/Portable_building.

Motazedian, D. 2017. Identification of seismic zones in Canada and their corresponding threshold values for spectral parameters provided in NBC 2015. Department of Earth Sciences, Carleton University.

Naumoski, N., Foo, S., and Saatcioglu, M. 2002. Seismic hazard assessment and mitigation for buildings' functional and operational components: a Canadian perspective. Prepared for Office of Critical Infrastructure Protection and Emergency Preparedness.

NRC. 1941. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1965. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1970. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1975. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1985. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1990. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 1995. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 2005. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NBC. 2010. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NRC. 2015. National Building Code of Canada. National Research Council of Canada, Ottawa, ON.

NEHRP. 1997. Recommended provisions for seismic regulations for new buildings and other structures. National Earthquake Hazard Reduction Program. Building Seismic Safety Council, Washington, D.C.

NIST. 2011. Standards of seismic safety for existing federally owned and leased buildings – ICSSC recommended practice 8 (RP 8). National Institute for Science and Technology, Gaithersburg, MD.

NRC. 1993a. Manual for screening of buildings for seismic investigation. National Research Council Canada, Ottawa, ON, Canada.

NRC. 1993b. Guidelines for seismic evaluation of existing buildings. National Research Council Canada, Ottawa, ON, Canada.

NZSEE. 2006. Assessment and improvement of the structural performance of buildings in earthquakes. Recommendations of a NZSEE study group on earthquake risk buildings. New Zealand Society for Earthquake Engineering.

Onur T., Seemann M., Halchuk S., Adams J. 2008. Probabilities of significant earthquake shaking in communities across Canada. The 14th World Conference on Earthquake Engineering, October 12-17, Beijing, China.

Onur, T., Ventura, C., and Liam Finn, W.D. 2005. Regional seismic risk in British Columbia — damage and loss distribution in Victoria and Vancouver. Canadian Journal of Civil Engineering. 32: pp. 361-371.

PWGSC. 2001. Real Property Service (RPS) policy on seismic resistance of PWGSC buildings. Public Works and Government Services Canada, Gatineau, QC.

Rainer, J. H., and Karacabeyli, E. 2000. Ensuring good seismic performance with platform-frame wood housing. Institute of Research in Construction, National Research Council of Canada, ISSN 1206-1220, December.

Ross, A.E., and Mahin, S.H. 1994. Steel moment resisting frames and the Northridge earthquake. Proceedings of the Sixth ATC 15-5 U.S. – Japan Workshop on the Improvement of Building Structural Design and Construction Practices, Victoria, BC, pp. 10.1 - 10.10.

Saatcioglu, M. 1994. Performance of reinforced concrete structures during the 1994 Northridge earthquake. Preliminary report on the Northridge, California, earthquake of January 17, 1994. Canadian Association for Earthquake Engineering, Vancouver, BC, pp. 226-250.

Sabelli, R., Roeder, C.W., J. F., Hajjar. 2013. Seismic design of steel special concentrically braced frame systems: a guide for practicing engineers. NEHRP Seismic Design Technical Brief No. 8, NIST GCR 13-917-24.

Shipp, J.G., Hart, G.C. 2011. Seismic design of wood and masonry buildings. Section 11. Retrieved on February 10, 2017 from http://extras.springer.com/2003/978-0-7923-7301-4/SDH2/Chapter%2011-Seismic%20Design%20of%20Wood%20and%20Masonry%20Buildings.pdf.

SIA. 2004. Pre-Standard SIA 2018: Examination of seismic safety of existing structures with regards to earthquakes, Swiss Society of Engineers and Architects, Zurich, Switzerland, 2004.

SSC. 1995. Seismic Safety Commission, State of California. Northridge earthquake: turning loss to gain. SSC Report No. 95-01, Sacramento, California, 1995.

Standards Australia. 2007. Structural design actions, Part 4: earthquake actions in Australia. Report AS 1170.4-2007, Council of Australian Standards, Sydney.

Sucuoglu, H., and Yazgan, U. 2003. Simple survey procedures for seismic risk assessment in urban building stocks. Seismic Assessment and Rehabilitation of Existing Buildings. NATO Science Series, Vol. IV/29 (Kluwer Academic Publishers, Dordrecht, 2003), pp. 97–118.

Sutley, E. J., and van de Lindt, J. W. 2016. Evolution of predicted seismic performance for wood-frame buildings. ASCE, Journal of Architectural Engineering, 22(3).

Sweny, A. 2012. Relationships between earthquake ground motions and Modified Mercalli Intensity. M.Sc. Thesis. University of Western Ontario.

TMS. 1994. Performance of masonry structures in the Northridge, California earthquake of January 17, 1994. The Masonry Society, Boulder, Colorado, pp.100.

Tremblay, R., Bruneau, M., Driver, R.G., Metten, A., Montgomery, C.J., and Rogers, C.A. 2010. Seismic design of steel structures in accordance with CSA-S16-09. Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering, July 25-29, Toronto, ON.

Tremblay, R., Dehghani, M., Fahnestock, L., Herrera, R., Canales, M., Clifton, C., and Hamid, Z. 2016. Comparison of seismic design provisions for buckling restrained braced frames in Canada, United States, Chile, and New Zealand. Structures. 8. (2016): 183-196.

Tremblay, R., Timler, P., Bruneau, M., and Filiatrault, A. 1995. Performance of steel structures during the 1994 Northridge earthquake. Can. J. Civ. Eng. 22: 338-360.

Tselentis G. A. and Danciu L. (2008). Empirical relationships between Modified Mercalli Intensity and engineering ground-motion parameters in Greece. Bull. Seismol. Soc. Am., p. 1863–1875.

UBC. 1970. Uniform building code. International Conference of Building Officials, Whitter, CA.

UBC. 1973. Uniform building code. International Conference of Building Officials, Whitter, CA.

UBC. 1976. Uniform building code. International Conference of Building Officials, Whitter, CA.

UBC. 1988. Uniform building code. International Conference of Building Officials, Whitter, CA.

UBC. 1997. Uniform building code. International Conference of Building Officials, Whitter, CA.

USGS (2005). Background information on ShakeMaps. Retrieved March 30, 2017 from https://escweb.wr.usgs.gov/share/shake2/about.html.

Uzumeri, S.M., Otani, S., and Collins, M. P. 1978. An overview of Canadian code requirements for earthquake resistant concrete buildings. Canadian Journal of Civil Engineering, 5: 427-441.

Varadharajan, S., Sehgal, V. K., and Saini, B. (2012). Review of different structural irregularities in buildings. Journal of Structural Engineering. Vol. 39, No. 5, Nov.

Wald D.J., Quitoriano V., Heaton T.H., Kanamori H. (1999). Relationships between peak ground acceleration, peak ground velocity, and Modified Mercalli Intensity in California. Earthq. Spectra, p. 557-564.

Wenk T. 2014. Seismic assessment based on cost-benefit considerations. Second European Conference on Earthquake Engineering and Seismology, Istanbul, 2014.

Worden C. B., Gerstenberger M. C., Rhoades D. A., Wald D. J. (2012). Probabilistic relationships between ground-motion parameters and Modified Mercalli Intensity in California. Bulletin of the Seismological Society of America, p. 204–221, 2012.

This page is intentionally left blank.

APPENDIX A EXISTING PRELIMINARY SEISMIC SCREENING METHODOLOGIES

This Appendix presents the existing preliminary seismic screening methodologies around the globe and key screening criteria.

A.1 Existing seismic screening methodologies

A.1.1 Canada

The existing Public Services and Procurement Canada (PSPC) Real Property Service (RPS) Policy regarding seismic resistance of *existing buildings* was issued in 2001 (PSPC, 2001). Seismic screening was only required to be conducted on *buildings* in zones of moderate to high seismicity, defined as zones where the effective seismic zone, Z_e, was greater than or equal to 2 (range 0 to 6). In other words, an effective seismic zone of 0 or 1 was considered as a zone with low seismicity, and *buildings* located in such zones were exempted from further *seismic risk* assessment. Z_e is a combination of the acceleration (Z_a) and velocity (Z_v) zones defined for each city in the National Building Code of Canada (NBC), and is equal to Z_v unless Z_a is greater than Z_v, in which case $Z_e = Z_v + 1$. In 2005, the NBC dropped the seismic zoning approach and has adopted the uniform hazard spectrum (UHS) approach (NEHRP, 1997), giving site-specific response spectral accelerations for numerous locations in Canada with a probability of exceedance of 2% in 50 years (equivalent to 2475-year return period).

A.1.2 United States

In the United States, the *Interagency Committee on Seismic Safety in Construction* (ICSSC) developed the 2011 Edition of the Standards of Seismic Safety for Existing Federally-Owned and Leased Buildings (NIST GCR 11-917-12), referred to in this report as NIST Standards (NIST, 2011). The NIST Standards provides a set of decision-making criteria outlining building characteristics and conditions that trigger a detailed seismic evaluation.

Based on NIST Standards (2011), *buildings* meeting the following conditions are exempt from seismic evaluation:

(1) All *buildings* located where S_{DS} <0.167 g and S_{D1} <0.067 g, namely Seismic Design Category (SDC) A; where S_{DS} and S_{D1} are the Design Earthquake Spectral Response Acceleration Parameters at short periods and at a 1.0 second period, respectively, as defined in ASCE 7-16. S_{DS} is more appropriate for short stiff *buildings* and S_{D1} is more appropriate for taller and more flexible *buildings*;

(2) All *buildings* located where $S_{DS} < 0.330$ g and $S_{D1} < 0.133$ g (SDCs A and B), unless designated for an occupancy-based performance objective (i.e., Risk Category VI in ASCE 7-16);

(3) Detached one- and two-family dwellings located where S_{DS} <0.4 g, unless designated for an occupancy-based performance objective;

(4) Building structures that are intended only for incidental human occupancy or that are occupied by persons for a total of less than 2 hours a day, unless designated for an occupancy-based performance objective;

(5) One-storey *buildings* of steel light frame or wood construction with areas less than 280 m^2, unless designated for an occupancy-based performance objective;

(6) *Buildings* scheduled for demolition;

(7) *Buildings* in foreclosure;

(8) Non-federally owned *buildings* leased by the federal government with temporary short-term leases;

(9) Non-federally owned *buildings* containing a total area leased by the federal government of less than 930 $m^{2,}$ where S_{DS} <0.50 g and S_{D1} <0.20 g; or

(10) *Buildings* designated by the agency as having a remaining useful life of, or fulfilling an agency need for, less than five years.

In addition to the full-building exemptions listed above, *buildings* (incorporating seismic provisions in their original design or having been seismically retrofitted) that qualify as benchmark *buildings* are deemed to comply with the structural evaluation and retrofitting provisions of NIST Standards. A benchmark *building* is a *building* constructed with deemed-adequate seismic design codes and expected to exhibit good seismic performance in the event of a damaging earthquake. The specified *benchmark NBC editions* by NIST Standards addresses only the structural scope of work. Unless otherwise exempt, non-structural components and geologic site hazards must still be considered, even for qualified benchmark *buildings*.

A.1.3 **New Zealand**

The New Zealand Society for Earthquake Engineering (NZSEE) released guidelines in 2017 for structural assessment of *buildings* subjected to earthquakes in New Zealand. The basic aim of the guidelines is to evaluate the earthquake performance of *existing buildings* and to specify whether they are earthquake prone. In the context of preliminary screening, the guidelines consist of an Initial Evaluation Procedure (IEP). This procedure is based on the qualitative assessment of the effects of any aspects of the structure and/or its parts that would be expected to reduce the performance of the building in earthquakes and thereby increase the life safety risks to occupants and/or have an adverse effect on neighbouring *buildings*. These building deficiencies are referred to as potential critical structural weaknesses (CSWs), and include construction type, presence of geologic hazards, age, quality of construction, and building condition. Furthermore, the NZSEE guidelines consider that there are some building types or categories whose potential earthquake-prone status can be predicted with some certainty, without initiating a formal seismic assessment process. For example, unreinforced masonry *buildings* where no previous strengthening has been carried out are regarded as potentially earthquake prone, and timber-framed *buildings* without

heavy roofs, and located on flat sites where the height of the ground floor above the ground is less than 600 mm are not considered to be earthquake prone.

According to the NZSEE guidelines, prior to undertaking formal initial seismic assessments (ISA) for all *buildings* in a community that are required to be considered under earthquake-prone building legislation, or within a portfolio, it may be appropriate to carry out an initial desktop exercise to prioritize *buildings* for assessment. This enables engineering resources to be utilized more efficiently since *buildings* that are likely to present a higher risk to life or are likely to be of high importance to the community in the aftermath of a large earthquake (e.g. hospitals, fire stations), are immediately flagged for further evaluation. However, the NZSEE guidelines caution that the prioritization process should not be seen as a substitute for using a formal initial seismic assessment process, such as an IEP, which is considered to be an important element of the overall ISA process.

A.2 Definition of low seismicity

This section provides an overview of seismic screening practices based on the definition of low *seismicity* for both structural and non-structural components.

A.2.1 **Canada**

A.2.1.1 Background

In Canada, 200 to 300 earthquakes are recorded each year (Uzumeri *et al.*, 1978), with an average of 14%, 27%, and 59% of these earthquakes occurring in Eastern, Western and Northern Canada, respectively. Until 1978, there was an average of one earthquake in Eastern Canada each decade with a magnitude greater than 6, and two each decade in Western Canada with magnitudes greater than 6.5.

Probabilistic strong seismic ground motion maps of Canada were first introduced in the NBC 1985 and remained unchanged in the NBC 1990. According to Basham *et al.* (1985), the probabilistic strong seismic ground motion maps of Canada in the NBC 1985 were developed based on ATC-3 (1978). As stated in ATC-3 (1978), the effective peak velocity (EPV) at long distances from an earthquake is related to the Modified Mercalli Intensity (MMI) scale. It was further assumed that the EPV logarithm would be linearly proportional to MMI.

The 1993 NRC screening manual was created to be compatible with the NBC 1990, in which Canada was divided into seven seismic zones (i.e., 0, 1, 2, 3, 4, 5, and 6) based on provided ground motion accelerations and velocities with a probability of exceedance of 10% in 50 years (equivalent to a return period of 475 years).

According to RPS Policy (PWGSC, 2001), an effective seismic zone of 0 or 1 was considered to be a zone with low seismicity, and *buildings* located in such zones were not required to be screened

for *seismic risk*s. Zones of moderate to high seismicity were defined as zones where the effective seismic zone, Z_e, was greater than or equal to 2.

In 2005, the NBC dropped the seismic zoning approach and adopted the uniform hazard spectrum (UHS) approach (NEHRP, 1997), giving site-specific response spectral accelerations for numerous locations in Canada with a probability of exceedance of 2% in 50 years (equivalent to 2 475-year return period).

A.2.1.2 Low Seismicity for Structure

In both the NBC 2005 and 2010, new *buildings* were exempted from seismic design provisions if located where design spectral acceleration at a 0.2-second period ($S(0.2)$) is less than or equal to 0.12 g. The design spectral response acceleration at a 0.2-second period ($S(0.2) = F_a(0.2)S_a(0.2)$) is comparable to the S_{MS} in ASCE 7-16. The main rationale for such exemption in the NBC 2010 is due to the fact that given the rare occurrence of severe earthquakes and consequently low earthquake hazard in the stable regions of Canada, the NBC 2010 exempted such areas – specifically those where $S(0.2)$ was less than 0.12 g – from the requirements of seismic design for all *buildings* regardless of importance category (Humar, 2015).

In the NBC 2015, however, the 0.12 g threshold value of $S(0.2)$ for exempting new *buildings* from seismic design requirements has been dropped. Humar (2015) explains the NBC 2015 revision in requiring seismic design for all over Canada, including low seismic zones, as follows:

(1) Large earthquakes can occur in stable areas, as demonstrated by the seismicity in other areas of the world; and,

(2) The eastern region of Canada has experienced considerable seismic activity.

The NBC 2015 requires seismic design to be carried out in all regions of Canada. Nevertheless, the code provides a simplified design procedure for areas of low hazard, yet without exempting buildings from seismic design requirements. The low hazard areas are defined as those where $I_E F_s S_a(0.2)$ is less than 0.16 g and $I_E F_s S_a(2.0)$ is less than 0.03 g, where I_E is the building importance factor, F_S is a site coefficient, and $S_a(0.2)$ is the spectral response acceleration at a 0.2-second period. Humar (2015) explains that the NBC 2015 threshold values of 0.16 g and 0.03 g are based on expert opinion. The value of 0.16 g for $I_E F_s S_a(0.2)$ implies that all *buildings* in a low seismic zone, such as Winnipeg, with the exception of important or post-disaster *buildings* located on poor soil, could be designed using the simplified procedure. This threshold similarly exempts *buildings* in some eastern locations from full seismic design where the seismic hazard is low, but still noticeable. *Buildings* in Calgary need detailed seismic design given that both the 0.16 g and 0.03 g thresholds are exceeded. Detailed seismic design is also required for some regions in British Columbia where the seismic hazard curve is quite flat and the 0.2-second period value is quite low, yet the 2-second period value is higher than 0.03 g.

Based on the preceding discussion regarding the definition of low seismicity in the NBC 2005 and subsequent editions, a direct correlation between MMI levels and the specified threshold values for spectral acceleration parameters is not evident. The importance of using such a correlation is that one can define the threshold values for design response acceleration parameters of low seismic regions by their correlation with MMI levels V (no *building damage*), and VI (threshold for non-structural damage) (ASCE 7-16). The definition of low seismicity in view of such correlation is especially of high interest for the case of *existing buildings*. One reason for such high importance is explained by ASCE 41-17 as follows:

> *"For new construction, a high level of safety and confidence in the seismic design can be safeguarded by any new building code provision due to more complete design flexibility and construction quality control at marginal additional cost. Nevertheless, the constraints of existing buildings often make the same level of performance reliability as a new building much more costly. Therefore, it would seem unreasonable to evaluate an existing building with the same scale as is used in case of a new building. This is especially of high interest in the case of an existing building located in a zone corresponding to low MMI levels (i.e., V) where no building damage is expected."*

A.2.1.3 Low seismicity for non-structural components

The PSPC RPS Policy regarding seismic resistance of non-structural components refers to "Guideline on Seismic Evaluation and Upgrading onof Non-Structural Building Components" (PSPC, 1995), which was superseded by the draft CSA S832 "Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings" in 2001. The current CSA S832-14 edition was designed to be compatible with the NBC 2010.

The PSPC Guideline referred to the NBC 1995, and did not require seismic design for non-structural components when either the velocity-related zone, Z_v, or the acceleration-related seismic zone, Z_a, is equal to or less than 1, and the foundation factor, F, is less than 1.3. Low seismicity is defined when the product $v \times F$ is less than or equal to 0.1, where v is the zonal velocity ratio and F is the foundation factor.

As stated earlier, in both NBC 2005 and 2010, all new *buildings* were exempt from seismic design provisions if located where design spectral response acceleration at a 0.2-second period ($F_a S_a(0.2)$) is less than or equal to 0.12 g. This exemption includes all types of both structural and non-structural components. The specified exemption of all non-structural components in the NBC 2005 and 2010 is comparable to the exemption of all non-structural components in FEMA E-74 (FEMA, 2012) for buildings located in an area with minimal level of shaking (which corresponds to MMI V as the threshold for no damage). ASCE 7-16 exempts SDCs A and B with the exception of parapets supported by bearing walls or shear walls. Parapets are not exempt because they can fail and pose a significant falling hazard even at low-shaking levels (commentary

C13.1.4 in ASCE 7-16). However, ASCE 7-16 exempts all non-structural components in SDC A from seismic design requirement (Section 11.7 in ASCE 7-16). CSA S832-14 uses the NBC 2010 threshold value of $F_a S_a(0.2) = 0.12\ g$ to determine the *Risk Index* threshold *RI* = 16 for *seismic risk* assessment mitigation of hazardous operational and functional components (OFCs).

As well, in the NBC 2005 and 2010, for all new *buildings* other than post-disaster *buildings*, certain non-structural components (except exterior and interior walls, parapets, exterior and interior ornamentations and appendages, towers, chimneys, diaphragms, smokestacks, and penthouses) were exempted from seismic design provisions if located where $I_E F_a S_a(0.2) \leq 0.35$ g. Not exempting certain non-structural components from seismic design is based on previous experience that has shown that these items can fail and pose a significant falling hazard, even at low shaking levels (ASCE 7-16). The preceding specified exemption of non-structural components in the NBC 2005 and 2010 is comparable to the exemptions of non-structural components in SDC B in ASCE 7-16, as well as the FEMA E-74 exemption of non-structural components in buildings located in an area with Low level of shaking (corresponding to MMI VI as the threshold for light non-structural damage).

CSA S832-14 used the 0.35 g threshold value of $F_a S_a(0.2)$ in the NBC 2010 to recommend mitigation techniques for parapets by suggesting a maximum height-to-thickness (h/d) ratio of unreinforced masonry parapets with insufficient anchorage capacity to reduce likelihood of overturning and possibility of falling debris. Maximum h/d are provided in Table A.1, as follows:

Table A.1: Height-to-thickness limits for unreinforced masonry parapets (from CSA S832-14)

$(h/d)_{max}$	$F_a S_a(0.2)$
4	$F_a S_a(0.2) \leq 0.2\ g$
2.5	$0.2\ g < F_a S_a(0.2) \leq 0.35\ g$
1.5	$0.35\ g < F_a S_a(0.2) \leq 0.55\ g$
1.0	$0.55\ g < F_a S_a(0.2)$

CSA S832-14 also sets height-to-thickness ratio limits for unreinforced masonry walls in new facilities as follows:

Table A.2: Height-to-thickness limits for unreinforced masonry walls (from CSA S832-14)

$(h/d)_{max}$		$F_aS_a(0.2)$
Wall located on:		
Top storey	Other storeys	
-	20	$F_aS_a(0.2) \leq 0.35$
14	16	$0.35 < F_aS_a(0.2) \leq 0.55$
9	13	$0.55 < F_aS_a(0.2)$

However, in the NBC 2015, the 0.12 g threshold value of $F_aS_a(0.2)$ for exempting new *buildings* from seismic design requirements has been dropped. The NBC 2015 provides a simplified design procedure for areas of low hazard, yet with no exemption for any building from seismic design requirements (both structural and non-structural components). The low hazard areas are defined as those where $I_EF_sS_a(0.2)$ is less than 0.16 g and $I_EF_sS_a(2.0)$ is less than 0.03 g. The NBC 2015 does not directly exempt non-structural components from seismic design requirements when located in areas of low seismic hazard ($I_EF_sS_a(0.2) \leq$0.16 g and $I_EF_sS_a(2.0) \leq$0.03 g), however, it only requires that cantilever parapets, other cantilever walls, exterior ornamentations and appendages, towers, chimneys and penthouses be designed for seismic forces. The specified thresholds for areas of low seismic hazard are comparable to the exemptions of all non-structural components in SDC A in ASCE 7-16, as well as FEMA E-74 for buildings located in an area with minimal shaking level (corresponding to MMI V as the threshold for no damage).

Regarding the exemption of non-structural components, in the NBC 2015, there are some other minor revisions with respect to previous editions. According to Clause 4.1.8.18 in the NBC 2015, for *buildings* other than *post-disaster buildings*, seismically isolated *buildings* and *buildings* with supplemental energy dissipation systems, non-structural components (except floor-mounted steel pallet storage racks and those mentioned in the NBC 2010) are exempt from seismic design provisions if located where $I_EF_aS_a(0.2) \leq$0.35 g.

A.2.2 **United States**

A.2.2.1 Background

Since 2012, the International Building Code (IBC) has specified the risk-targeted maximum considered earthquake (MCE_R) ground motions for designing new *buildings* and other structures. The method provides a framework for assessing hazard based on a probability of collapse rather than ground motion exceedance probabilities.

The older, uniform seismic hazard maps of MCE specified shaking that had a 2% chance of exceedance in 50 years. The risk-targeted ground motion maps take into account regional differences in the shape of the seismic hazard curve. The distinction between the MCE and MCE_R is that a risk-targeted design aims for a consistent probability of life-threatening damage during a

particular period of time, as opposed to a consistent probability of exceedance earthquake ground motion. MCE_R ground motions are higher than MCE in California and are generally smaller than MCE in other parts of the United States (ASCE, 2016).

Luco *et al.* (2007) offer a method to relate collapse probability conditioned on a given level of shaking to frequency of collapse-causing earthquakes, for a particular building and a particular location. Their purpose was to select a level of shaking at that site (which is named MCE_R) such that, if the building had a 10% collapse probability conditioned on the MCE_R, it would also have a 1% collapse probability in 50 years, considering all levels of shaking that could occur at that location, and how the probability of collapse varies with shaking.

ASCE 7-16 also considers the equivalent Modified Mercalli Intensity (MMI) scale in developing the ground motion limits and design requirements for the various SDCs. According to ASCE 7-16 Commentary, there are correlations between the qualitative MMI scale and quantitative characterizations of ground motions, as summarized in Table A.3. MMI level VI is generally considered as the threshold for non-structural damage, and MMI VII is considered as the threshold for structural damage (Onur *et al.*, 2008). Therefore, no *building damage* (structural and non-structural) is expected for a building located in a zone with MMI level V.

Table A.3: Modified Mercalli intensity vs. damage level (from ASCE 7-16)

MMI scale	Damage level
V	No damage
VI	Light non-structural damage
VII	Hazardous non-structural damage Light structural damage
VIII	Hazardous damage to susceptible structures
IX	Hazardous damage to robust structures

According to the ASCE 7-16 commentary, the upper limit for SDC A (i.e., the lower limit for SDC B) was set at roughly one-half of the lower threshold for MMI VII (i.e., the upper limit for MMI VI) and the lower limit for SDC D was set at roughly the lower threshold for MMI VIII. However, the lower limit for SDC D was more consciously established by equating the design value (two-thirds of the MCE_R) to one-half of what had been the maximum design value in building codes over the period of 1975 to 1995. Table A.4 presents the correlation in ASCE 7-16 between SDC and the design spectral acceleration parameters for ordinary occupancies:

Table A.4: Design spectral acceleration parameters for ordinary occupancy (from ASCE 7-16)

SDC	Design spectral acceleration parameter threshold values	
A	$S_{DS} < 0.167\ g$	$S_{D1} < 0.067\ g$
B	$0.167\ g \leq S_{DS} < 0.33\ g$	$0.067\ g \leq S_{D1} < 0.133\ g$
C	$0.33\ g \leq S_{DS} < 0.50\ g$	$0.133\ g \leq S_{D1} < 0.20\ g$
D	$0.50\ g \leq S_{DS}$	$0.20\ g \leq S_{D1} < 0.75\ g$
E	–	$S_{D1} > 0.75\ g$

According to ASCE 7-16, the design response acceleration parameters S_{Ds} and S_{D1} are considered to be sufficient when adjusted for site effects to define an entire response spectrum for the period range of importance for most *buildings* and structures, using the generic shape of the MCER response spectrum.

According to ASCE 7-16 commentary, at the MCER level, SDC A structures should not experience motions that are normally destructive to structural systems, while the MCER level motions for SDC D structures can destroy susceptible structures.

It is evident in Table A.4 that no threshold value is defined for S_{DS} for a building in SDC E. This is explained by FEMA 303 (FEMA, 1997) as follows:

> *For the purpose of determining whether a structure is located in a region that is very close to a major active fault or not, FEMA 303 uses a trigger of a mapped MCE spectral response acceleration at 1.0 second periods, S_{D1}, of 0.75 g or more regardless of the structure's fundamental period. The mapped short period acceleration, S_{Ds}, is not used for this purpose because short period response accelerations S_{Ds} do not tend to be affected by near-source conditions as strongly as do response accelerations at longer periods.*

Design spectral acceleration parameters (i.e., S_{DS} and S_{D1}) are defined as follows:

$$S_{DS} = 2/3\, S_{MS} \tag{A.1}$$

$$S_{MS} = F_a S_s \tag{A.2}$$

$$S_{D1} = 2/3\, S_{M1} \tag{A.3}$$

$$S_{M1} = F_v S_1 \tag{A.4}$$

where:

S_S = the mapped MCER spectral response acceleration parameter at short periods (at 0.2 seconds)

S_1 = the mapped MCER spectral response acceleration parameter at long periods (at 1.0 second)

F_a = short-period site coefficient (at 0.2 seconds)

F_v = long-period site coefficient (at 1.0 second)

The SDCs in ASCE 7-16 are adopted by ASCE 41 (ASCE, 2017), in which the levels of seismicity include Very Low, Low, Moderate, and High (corresponding to SDCs A, B, C, and D, respectively).

According to ASCE 7-16, earlier editions of the NEHRP provisions used the peak velocity-related acceleration A_v to determine a *building*'s seismic performance category. However, this coefficient does not adequately represent the *damage* potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 NEHRP provisions adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site effects for this purpose.

It is noteworthy that the assumed reference *Site Class* in ASCE 7-16 is Site Class B, while in the NBC 2015, it is Site Class C. For comparison purposes, S_{MS} and S_{M1} would be compared with the $S(0.2)$ and $S(1.0)$ parameters in the NBC 2015.

A.2.2.2 Low seismicity for structure

The MMI correlation with response spectral acceleration values is of especially high interest for defining low seismicity, as it could potentially be used as one justification for exempting *existing buildings* located in a low seismic zone from further detailed *seismic risk* screening. The definition of low seismicity can be reasonably linked to MMI levels V and VI, which represent the perceived shaking level corresponding to no damage to both structural and non-structural components and no to light damage to non-structural components.

The Design Spectral Acceleration Parameters threshold values corresponding to SDC A are used in the 2011 edition of the Standards of Seismic Safety for Existing Federally Owned and Leased *Buildings* (NIST GCR 11-917-12), referred to as the NIST Standards (NIST, 2011), which recommends that *existing buildings* located in zones where S_{DS} < 0.167 g and S_{D1} < 0.067 g may not all require screening and evaluation. This exemption is based on ASCE 7-16 where no systematic seismic design for new *buildings* in SDC A is required (i.e., nominal structural seismic force equal to 1% of building weight is dictated, yet no non-structural seismic design is required).

The preceding specified threshold values for S_{DS} and S_{D1} correspond to SDC A in ASCE 7-16, which is related to *buildings* in all Risk Categories (i.e., I, II, III, and IV). Therefore, the specified exemption is applicable to all *existing buildings* regardless of risk category; consequently, one

could expect no structural or non-structural damage in a building located in zones where S_{DS} < 0.167 g and S_{D1} < 0.067 g (corresponding to MMI V).

The S_{DS} and S_{D1} threshold values corresponding to SDC B are also used by NIST Standards for exempting all *buildings* located in zones where S_{DS} < 0.330 g and S_{D1} < 0.133 g (corresponding to MMI VI) unless designated for an occupancy-based performance objective (i.e., risk category IV).

A.2.2.3 Low seismicity for non-structural components

Past earthquakes have shown that damage to *buildings'* non-structural components usually cause more injuries, fatalities, and property and financial losses than those imposed by structural damage (Naumoski *et al.*, 2002). There have been many cases where seismic events caused only minor structural damage, yet *buildings* have been deemed to be unsafe and unusable due to extensive damage to their non-structural components (Naumoski *et al.*, 2002). Onur *et al.* (2005) also emphasize that in a number of recent earthquakes, it has been observed that non-structural damage usually dominates the economic loss and frequently causes casualties. Therefore, estimating non-structural damage has become of great importance in *seismic risk* and loss estimation studies.

Building non-structural components include: architectural components, mechanical and electrical equipment and building contents. Non-structural components failure and resulting debris can potentially affect the performance of vital facilities (e.g., hospitals, power stations etc.).

The extent of seismic design provisions in ASCE 7-16 for non-structural components of new *buildings* depends on the building's SDC, and indirectly on the MMI level:

For SDC A: all non-structural components are exempt from seismic design provisions of ASCE 7-16.

FEMA E-74 exemption criteria is consistent with that in ASCE 7-16, where all non-structural components in all new *buildings* are exempt from seismic retrofit if the building is located in an area with minimal level of shaking. Minimal level of shaking in FEMA E-74 is based on the International Residential Code (2006), which is equivalent to SDC A in ASCE 7-16.

For SDC B: all non-structural components except for parapets and other architectural components with component importance factor (I_p) greater than 1 are exempt from seismic design provisions of ASCE 7-16.

The rationale behind not exempting certain non-structural components from seismic design provisions comes from previous experience that has shown that these items can fail and pose a significant falling hazard, even at low shaking levels (ASCE 7-16).

FEMA E-74 exemption criteria is consistent with that in ASCE 7-16, where all non-structural components except parapets and exterior unreinforced masonry walls in all *buildings* are exempt

from seismic retrofit if the building is located in an area with low level of shaking. Low level seismic zone in FEMA E-74 is equivalent to SDC B in ASCE-16.

In ASCE 41-17, parapets and cornices with the following conditions are required to be evaluated and retrofitted:

- Unreinforced masonry parapets with aspect ratio (parapet height above its anchorage level to floor, divided by its thickness) > 1.5
- Reinforced masonry/concrete parapet with aspect ratio > 3.0
- Cornices/ledges made of stone, terra cotta, or brick, unless supported by a steel / reinforced concrete structure
- Sculptures and ornamental features made of stone, terra cotta, masonry, or concrete with aspect ratio > 1.5

For SDC C: mechanical/electrical components are exempt from the seismic design provisions in ASCE 7-16 when all of the following conditions are met:

(i) Component importance is $I_p = 1.0$.

(ii) Component is positively attached to the structure.

(iii) Component weight is equal to or less than 89 N, or 73 N/m in case of distributed system.

FEMA E-74 exemption criteria is comparable with that in ASCE 7-16, where all non-structural components except architectural components in all non-essential *buildings* are exempt from seismic retrofit if the building is located in an area with moderate level of shaking. Moderate level of shaking in FEMA E-74 is equivalent to SDC C in ASCE 7-16.

For SDCs D, E, and F: discrete mechanical/electrical components that are positively attached to the structure are exempt from seismic requirements of ASCE 7-16, only when all of the following conditions are met:

(i) Component importance is $I_p = 1.0$.

(ii) Component is positively attached to the structure, and its weight is equal or less than 1.78 kN. The centre of component mass distance to adjacent floor level is less than 1.22 metres. Flexible connections are provided between the component and associated accessories such as ductwork, piping, and conduit.

(iii) Component weight is equal or less than 89 N, or 73 N/m for distributed systems.

In addition, distribution systems in the exceptions for conduit, cable tray, raceways, duct systems, as well as piping and tubing systems are exempt from seismic requirements of ASCE 7-16.

FEMA E-74 exemption criteria are comparable with that in ASCE 7-16, where in high level of shaking, all mechanical/electrical components in non-essential *buildings* are exempt from seismic

requirements if they weigh less than 1.78 kN and are mounted at a height of 1.22 meters or less above the floor or, if elevated, weigh less than 89 N. Distributed systems in non-essential facilities are also exempt from bracing or anchoring if they weigh less than 73 N/m and are provided with flexible connections. High level of shaking in FEMA E-74 is equivalent to SDC D, E, and F in ASCE 7-16.

A.2.3 **New Zealand**

Experience indicates that chimneys, specifically, can be vulnerable and potentially pose life-threatening falling hazards, even at low levels of earthquake shaking, especially if they are unreinforced or poorly braced to the building (NZSEE, 2017).

NZSEE (2017) recognizes a chimney as potentially seismic prone, if either:

- not restrained by the roof structure, or other fixing, at the roofline, or
- meeting all of the following criteria:
  - constructed of unreinforced masonry/concrete, and;
  - the height-to-plan dimension (h/d) ratio of the chimney in the direction being considered is more than the values in Table A.5:

Table A.5: Height-to-plan dimension limits for unreinforced chimneys (from NZSEE, 2017)

$(h/d)_{max}$	$Z \times R$
1.5	$0.3 \leq Z \times R$
2	$0.2 < Z \times R < 0.3$
3	$Z \times R \leq 0.2$

where Z is the hazard factor as 0.5 times the magnitude-weighted 5% damped response spectrum acceleration for a 0.5-second period for *Site Class* C (shallow soil) with a return period of 500 years, and R is a modification factor for different return periods), and

(i) any one or more of the following applies:
  - Any possibility of the chimney falling on an egress route, entrance way, over a boundary (including over a street frontage), over any public/private access way or more than 2 m down onto an adjoining part of the building, or
  - Roofing material comprised of concrete masonry, clay tiles or other brittle material, unless suitable sheathing was laid within the ceiling space to prevent the roofing material and collapsed chimney from falling through the ceiling.

A.3 *Benchmark NBC editions* of *model building types*

When a new edition of the NBC is released, one key question to owners of existing *buildings* is whether or not their *existing buildings* can still be considered seismically adequate. To answer this

question, one generally needs to have a deep understanding of the extent and significance of changes in the seismic design/detailing provisions of the NBC over its previous editions.

The year in which substantial improvements in seismic code requirements were adopted and enforced is referred to as benchmark year (FEMA P-154). Any building designed and constructed to the applicable benchmark year or after is referred to as *post-benchmark building*. Table A.6 summarizes the specified benchmark years in the first (1988), second (2002), and third edition of FEMA P-154 (2015) and in NIST Standards for different *building types*.

Table A.6: Benchmark years for different *building types*

Building type	**Benchmark year**			
	FEMA 154 (1988)	FEMA 154 (2002)	NIST (2011)	FEMA P-154 (2015)
W1	1949	1976	1976	1976
W1A	1949	-	1997	1997
W2	1949	1976	1976	1976
S1	1976	1994	1994	1994
S2	1988	1988	1997	1997
S3	NO BM	NO BM	1992 (FEMA 178) 1998 (FEMA 310 / ASCE 31) 2000 (IBC / FEMA 356 / ASCE 41)	NO BM
S4	1976	1976	1994	1994
S5	NO BM	NO BM	1998 (FEMA 310 / ASCE 31) 2000 (IBC / FEMA 356 / ASCE 41)	NO BM
C1	1976	1976	1994	1994
C2	1976	1976	1994	1994
C3	NO BM	NO BM	1998 (FEMA 310 / ASCE 31) 2000 (IBC / FEMA 356 / ASCE 41)	NO BM
PC1	1973	1997	1997	1997
PC2	NO BM	NO BM	1992 (FEMA 178) 1998 (FEMA 310 / ASCE 31) 2000 (IBC / FEMA 356 / ASCE 41)	NO BM
RM1	1976	1997	1997	1997
RM2	1976	1976	1994	1994
URM	NO BM	1991	URM: UBC: 1991 URMA: NO BM	NO BM
MH	-	-	-	1995 (California) 2009 (Other states)

As it is evident in Table A.6, major changes have been made in the specified benchmark years of different *building types* in the first 1988 edition of FEMA 154 and its subsequent 2002 and 2015 editions. This can be attributed to the revised seismic design provisions in seismic codes and standards based on the lessons learned from seismic performance of different building types after major earthquakes in the United States (e.g., 1989 Loma Prieta and 1994 Northridge earthquakes) and other countries (e.g., 1995 Kobe earthquake).

FEMA P-154 identified benchmark years and provided *post-benchmark buildings* with some credit in the process of quantitative structural scoring, acknowledging their ductility capacity for energy dissipation. Because *building damage* in certain building types, including steel frame with infill masonry walls (SIW), concrete frame with infill masonry walls (CIW), and unreinforced masonry walls (URM), is a function of lateral force instead of displacement, ductility capacity for inelastic energy dissipation is not acknowledged in model *building* codes and relevant standards. As a result, *benchmark NBC editions* are not applicable for these *building types*.

It is noted that the specified benchmark years in FEMA P-154 and NIST Standards serve different purposes. While a building qualified as a *post-benchmark building* according to FEMA P-154 is provided with certain benefits in the quantitative structural scoring process for acknowledging inelasticity and energy dissipation capacity, it does not warrant the *building's* exemption from further *seismic risk* assessment. However, a *building* qualified as a *post-benchmark building* by NIST Standards is deemed to comply with the structural seismic evaluation provisions in NIST Standards. A structural seismic evaluation exemption is warranted given that stringent *building* height and *seismicity* restrictions have been enforced in *building* code provisions.

In Canada, the NBC is the model *building* code, which typically references a number of standards issued by various standards development organizations (e.g., Canadian Standards Association – CSA). While the NBC provides design loads for *buildings* (i.e., dead, live, wind, seismic, etc.), the standards development organizations (e.g., CSA in Canada) develop and maintain standards for design of structures constructed with different material types (i.e., steel, concrete, timber, masonry, etc.). The NBC adopts these standards by referencing them and making only a handful of modifications. A standard by itself has no legal standing of its own. It acquires legal authority within a jurisdiction through a two-step adoption process:

- Adoption or reference of the standard into the NBC
- Adoption of the NBC by jurisdictions

It should be noted that the term "benchmark year" in FEMA P-154 could be confusing in some cases. For example, an S1 (steel moment frame) *building* could be built in 1995 but designed in accordance with the UBC 1991. Given that the S1 *building's* benchmark year is 1994, the screener may identify the *building* as a *post-benchmark building* based on the year built. To avoid the

confusion and possible mistake, in the *Level 1 – PST*, *benchmark NBC edition* is used, rather than benchmark year. The *benchmark NBC edition* refers to an applicable NBC edition in which significantly improved seismic requirements were adopted and enforced. If municipal *building* codes or provincial *building* codes were used to design *buildings*, the applicable NBC editions on which these *building codes* were based should be identified.

Determining the *benchmark NBC edition* is a complex task. The key factor in specifying the appropriate *benchmark NBC edition* for each building type is finding the edition of the code in which major improvements were made to its seismic design provisions. A summary table of the evolution of seismic provisions in both Canada and the U.S. is presented in APPENDIX C. Given the close technical co-operation that exists between Canadian and American engineers, many aspects of the Canadian code requirements are essentially similar to those in American codes (Uzumeri *et al.*, 1978). Therefore, after finding out the edition of the applicable code in which major seismic design improvements were made for the United States, the corresponding changes will be tracked down in Canada in order to determine the applicable *benchmark NBC edition* for different *building* types in Canada.

Table A.7 maps the various building types used in FEMA P-154 and the *Level 1 – PST*. Justification of building type mapping in the 1993 NRC screening manual can be found elsewhere (NRC, 1993a). It is noted that cold-formed steel (CFS) building type is considered a new model building type in the *Level 1 – PST*. This is consistent with the latest ASCE 41-17 edition, where the CFS building type was first introduced as a new common building type.

Table A.7: A mapping of building types in FEMA P-154 and the Level 1 – PST

FEMA building type	NRC building type	Description of building characteristics
W2	WLF	Engineered Wood Light Frame *buildings* of up to 6 storeys in *building* height, or having a *building* area exceeding 600 m^2
W2	WPB	Engineered Wood Post-and-Beam *buildings* which are covered by Part 4 of the NBC
S1	SMF	Steel Moment Frame
S2	SBF	Steel Braced Frame
S3	SLF	Steel Light Frame
S4	SCW	Steel frame with Concrete shear Walls
S5	SIW	Steel frame with Infill masonry shear Walls
C1	CMF	Concrete Moment Frame
C2	CSW	Concrete Shear Walls
C3	CIW	Concrete frame with Infill masonry shear Walls
PC1	PCW	Precast Concrete Walls
PC2	PCF	Precast Concrete Frames
RM1	RML	Reinforced Masonry bearing walls with Light wood or metal deck diaphragms
RM2	RMC	Reinforced Masonry bearing walls with Concrete diaphragms
URM	URM	Un-Reinforced Masonry bearing wall buildings
--	CFS	Cold-Formed Steel buildings
MH	MH	Manufactured Homes

In the following, justification(s) for benchmark year identification in FEMA P-154 are presented. The specified benchmark years in the U.S. are then mapped into Canadian context to determine the *benchmark NBC editions* for the *building types* in the *Level 1 – PST*.

A.3.1 Benchmark NBC editions for wood buildings

Wood buildings consist of two model building types: (1) engineered wood light frame (WLF) *buildings* of up to 6 storeys in *building* height, or having a *building* area exceeding 600 m^2, and (2) engineered wood post-and-beam (WPB) *buildings* which are covered by Part 4 of the NBC. The seismic design of wood *buildings* in Canada is performed in accordance with the requirements of the NBC and the "Engineering Design in Wood" CSA standard. CSA O86 is intended for use in designing wood structures, and provides criteria for the structural design and appraisal of structures or structural elements made from wood or wood products, including graded lumber, glued-laminated timber, unsanded plywood, oriented strandboard, composite building components, shear walls and diaphragms, timber piling, pole-type construction, prefabricated wood I-joists, structural composite lumber products, preserved wood foundations, and their

structural connections. This standard employs the limit states design method. The tenth edition of CSA O86 was published in 2014.

FEMA P-154 specified 1976 UBC as benchmark code edition for WPB buildings. This is due to the significant changes made in the 1976 UBC based on the experience gained and the structural damage observed after the 1971 San Fernando earthquake. In Canada, seismic provisions for the design of shear walls and horizontal diaphragms first appeared in the 1989 edition of the CSA O86 (CSA, 1989), which was referenced by the NBC 1990 (NRC, 1990). The Canadian Wood Council (CWC, 2003) claimed noted that wood structures (including WLF and WPB) have sufficient seismic resistance in moderate earthquakes without substantial damage and major earthquakes without collapse if designed according to the NBC 1995. The NBC 2005 introduced provisions to mitigate collapse due to soft-storey and other types of building irregularities in wood structures. This NBC edition also introduced restrictions on seismic analysis methods for buildings with certain irregularities (DeVall, 2003; Varadharajan et al., 2012).

It is important to note that the height of wood buildings was limited to four storeys or less in the NBC 2010 and earlier editions. Recent advances in wood construction technology, including the development of innovative wood products, have led to the adoption of mid-rise wood buildings (five to six storeys) in Canada. Design requirements for mid-rise (five to six storeys) combustible construction were first introduced in the 2014 edition of CSA O86 (CSA, 2014a). These provisions were adopted in the NBC 2015, which permits the construction of six-storey residential, business and personal service buildings using traditional combustible construction materials.

It is important to note that the height of wood *buildings* was limited to four storeys or less in the NBC 2010 and prior editions. Recent advances in wood construction technology, including the development of innovative wood products, have led to the adoption of mid-rise wood *buildings* (five to six storeys) in Canada. More than 250 mid-rise wood *buildings* have been constructed in British Columbia since 2009 when the British Columbia building code allowed construction of five- and six-storey wood *buildings*. Shortly thereafter, other provincial jurisdictions, including Quebec in 2013, and Ontario and Alberta in 2015, have allowed construction of mid-rise wood *buildings*. Design requirements for mid-rise (five to six storey) combustible construction were first introduced in CSA O86-14. These provisions were adopted in the NBC 2015, which permits the construction of six-storey residential, business and personal service *buildings* using traditional combustible construction materials (NBC 2015).

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for WLF buildings with up to four storeys, and the NBC 2015 was chosen as the *benchmark NBC edition* for WLF buildings with more than four but no more than six storeys. For WPB buildings, the NBC 2005 is chosen as the *benchmark NBC edition*.

A.3.2 **Benchmark NBC editions for concrete buildings**

Seismic design of concrete *buildings* in Canada is performed in accordance with the requirements of the NBC and CSA A23.3. CSA A23.3 is intended for use in designing concrete structures for *buildings* in conjunction with CSA A23.1/A23.2, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete, and CSA A23.4, Precast Concrete – Materials and Construction. The current version of CSA A23.3 is the sixth edition published in 2014. In the following section, the determination of benchmark years for different types of concrete *buildings* is discussed in detail.

A.3.2.1 Concrete moment frame (CMF)

This *building type* consists of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consist of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. Two construction subtypes fall under this category:

- Non-ductile reinforced-concrete frames without reinforced infill walls
- Ductile reinforced-concrete frames

The most prevalent of these is non-ductile reinforced concrete frame structures without reinforced infill walls built between about 1950 and 1972. In many regions of Canada, this type of construction continues even today.

In the first and second editions of FEMA 154, the benchmark year for C1 (equivalent to CMF) was specified in 1976. Following the 1971 San Fernando earthquake, it was concluded that additional transverse reinforcing was required in high-seismicity regions. The new requirements first appeared in the 1976 edition of the UBC. In the third edition of FEMA P-154, the benchmark year was moved to 1994. Following the 1994 Northridge earthquake, additional code requirements for improving ductility were added to the 1994 UBC.

The first Canadian special provisions for seismic design of reinforced concrete members were introduced in CSA A23.3-73 (Adebar *et al.*, 2015), which generally mirrored the seismic requirements for moment-resisting frames in ACI 318-71. Many examples of collapse in slab-column CMF structures were reported by Mitchell *et al.* (1995) from past earthquakes (e.g., 1985 Mexican earthquake). These were caused by punching shear failures due to the combined effects of direct shear and reversed cyclic shear caused by moment transfer from the slabs to the columns. Due to the brittle nature of punching shear failures, this type of failure often leads to a progressive collapse. CSA A23.3-84 imposed additional requirements to improve the seismic performance of CMF *buildings*. The new requirements included minimum amount of carefully detailed bottom reinforcement for structural integrity and new special detailing requirements for columns in nominally ductile frames where plastic hinges were expected to develop and form diagonal tension

cracks (Mitchell and Cook, 1984). While CSA A23.3-84 introduced detailing requirements for columns in nominally ductile frames where plastic hinges were expected to develop in the columns, CSA A23.3-94 introduced an additional requirement that each column must have a shear capacity such that: 1) the nominal moment resistance of the column can be developed at the ends of the column, or 2) the column shear is increased to the level corresponding to $R = 1.0$.

After the 1994 Northridge earthquake, it was observed that many columns only designed for gravity loads had failed (Mitchell *et al.*, 1995). This failure type was due to the fact that they were not detailed to behave as ductile members (e.g., insufficient amounts of shear and confinement reinforcement). CSA A23.3-94 required that frame members not considered as part of the SFRS be capable of deforming laterally to mitigate failures such as those observed at Northridge.

In the NBC 2005, different height restrictions were introduced for reinforced concrete SFRS, including CMF *buildings*. These restrictions were based on ductility level, seismicity, site class, and building importance. The NBC 2005 and its subsequent editions also introduced the influence of inherent over-strength in different structural systems (Mitchell et al., 2003), along with the importance of ductility in seismic design. It is important to note that, since the NBC 2015, two-way slabs without beams have been explicitly considered as one type of concrete SFRS.

Based on the preceding discussion, the NBC 2015 is chosen as the *benchmark NBC edition* for CMF buildings constructed using two-way slabs without beams. The NBC 2005 is chosen as the *benchmark NBC edition* for other types of CMF buildings.

A.3.2.2 Concrete shear walls (CSW)

These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists or flat slabs. Buildings may also have steel beams, steel columns, cold-formed steel light-frame construction and concrete slabs for gravity framing. Seismic forces are resisted by cast-in-place concrete shear walls.

In the first edition of FEMA P-154, 1976 was determined as the benchmark year for C2 (equivalent to CSW) *buildings*. In the second and third editions of FEMA P-154, the benchmark year shifted to 1994. This is mainly due to the inclusion of brief comments in ACI 318-92 (referenced in the 1994 UBC) for the design of coupling beams in coupled shear walls with bi-diagonal reinforcement.

Based on their visit after the 1994 Northridge earthquake, Mitchell *et al.* (1995) reported that *buildings* with an adequate amount of properly distributed shear walls and diaphragm connections generally performed well. However, Moehle *et al.* (2011) reported the failure of some connections between diaphragms and walls in the 1994 Northridge earthquake. According to Saatcioglu (1994), shear failures involving both diagonal tension failures and diagonal crushing of the concrete in the coupling beams of concrete shear walls were observed in the 1994 Northridge earthquake. Furthermore, Mitchell *et al.* (1995) reported that many reinforced concrete shear wall *buildings*

with irregular geometries (e.g., irregularly placed shear walls, significant setback, large shear wall openings, etc., causing torsional eccentricities and weak storeys) experienced significant structural damage due to increased shear forces in shear walls, given the induced torsional effects.

Many of the deficiencies observed in U.S. *buildings* during the Northridge earthquake were also present in the then-current Canadian seismic design. Based on the performance of CSW *buildings* during the Northridge and similar earthquakes, significant improvements in the seismic design provisions for this type of structure were introduced in CSA A23.3 and the NBC. The seismic design provisions in CSA A23.3-04, which ensure concrete walls without confinement reinforcement have adequate flexural displacement capacity (ductility), have been completely revised from the provisions in both CSA A23.3-84 and CSA A23.3-94, and were later referenced in the NBC 2005. The new provisions require an estimate of the SFRS displacement demand due to the design earthquake. For ductile walls that are part of a system with good displacement control, the new provisions are less restrictive than the old provisions, while for moderately (nominally) ductile walls, the new provisions are considerably more restrictive than the old provisions. The effects of building irregularity and required seismic analysis methods (i.e., dynamic and equivalent static) were first introduced in the NBC 2005 (DeVall, 2003, and Varadharajan *et al.* 2013).

Unlike in the NBC 1995, in the NBC 2005 and subsequent editions, different height restrictions were introduced as with CMF buildings. (Please refer to A.3.2.1.) The NBC 2005 and subsequent editions limit the overall height of conventional concrete shear wall *buildings* to 30 m at sites where $I_E F_a S_a(0.2) > 0.75$ g. Although the NBC 1995 and its earlier editions have recognized the importance of ductility in seismic design (through R factor), only the NBC 2005 and its subsequent editions have implemented design approaches to consider the additional influence of the inherent over-strength in different structural systems (Mitchell *et al.*, 2003).

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for CSW buildings.

A.3.2.3 Concrete frame with infill masonry shear walls (CIW)

This is an older type of building construction that consists of a concrete frame with unreinforced-masonry or reinforced-masonry infill walls. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure.

There are three general types of infill walls. Participating infill walls in which there are openings or gaps between the masonry infill and the surrounding frame, but the infill walls are not tied or bonded to the frame. Composite action infill walls in which infill shear walls are tied and bonded to the frame to create a composite shear wall, where the infill forms the web and the frame columns form the flanges of the shear wall. Isolated infill walls in which the masonry is separated from the frame structure by a gap created by vertical movement joints along the ends and a horizontal

movement joint under the floor or beam above. Masonry infill is a non-load-bearing wall and cannot be treated as a shear wall.

No benchmark year is specified for C3 buildings (equivalent to CIW) in any of the three FEMA P-154 editions. This is attributed to the fact that these buildings are not expected to have appreciable ductile behaviour, in accordance with ASCE 7-16, in which the ductility capacity of C3 building type is not acknowledged. ASCE 7-16 has no limit for C3 *buildings* corresponding to seismic design categories (SDCs) A and B; however, it does not allow for the use of C3 *buildings* in higher SDCs. The NIST Standards specify a benchmark year of 2000 for this building type, if designed based on the IBC and FEMA 178, FEMA 356/ASCE 41. The NIST Standards specify a benchmark year of 1998 for this building type, if designed based on FEMA 310/ASCE 31. Specification of the benchmark year in the NIST Standards can be justified by its main purpose for exempting *post-benchmark buildings* from structural seismic evaluations.

According to Anderson and Brzev (2009), the following general design provisions for masonry infill walls were initially introduced in the second edition of CSA S304.1-04:

- Masonry infill shear walls to be designed for all in-plane and out-of-plane loads
- Infill walls to be designed for any vertical loads transferred through the frame
- Increased lateral stiffness of lateral load-resisting elements (consisting of masonry infill shear walls working with the surrounding frame), to be taken into account for distributing the applied loads to these elements
- If a diagonal strut model for the masonry infill shear wall is used, masonry infill frames shall be designed using a truss model.

In the NBC 2005 and subsequent editions, CIW buildings are identified as *other* concrete SFRS, and no ductility capacity is acknowledged. The concept of different structural systems having different levels of inherent over-strength was introduced for the first time in the NBC 2005 (Mitchell *et al.*, 2003). Furthermore, different height restrictions were introduced for *other* concrete SFRS, as with CMF buildings. (Please refer to A.3.2.1.) Therefore, the NBC 2005 is chosen as the benchmark year for the CIW building type.

A.3.2.4 Precast concrete walls (PCW)

PCW *buildings* are built using either precast shear walls or tilt-up wall panels. Most of them use the tilt-up construction method, therefore in this study, PCWs are treated as tilt-up *buildings*, which is consistent with building classification in the NRC screening manual (1993b) and FEMA P-154.

PCW *buildings* have precast concrete perimeter wall panels and often, interior walls, that are typically cast on site and tilted into place. More recently, wall panels have begun to be fabricated off-site and trucked in. The wall panels are welded together or held in place by cast-in-place columns or steel columns, depending on the region. The floor and roof beams are often glue-laminated wood or steel open-webbed joists attached to the tilt-up wall panels; these panels may

be loadbearing or non-loadbearing, depending on the region. Seismic forces are resisted by the precast concrete perimeter wall panels.

In the first edition of FEMA P-154, 1973 was determined as the benchmark year for PC1 *buildings* (equivalent to PCW). This is justified by the fact that seismic design requirements for PC1 buildings were improved following damage to PC1 buildings in the 1971 San Fernando earthquake (Housner, 1971). In the second and third editions of FEMA P-154, the benchmark year shifted to 1997. This is because major improvements were later adopted in the 1997 UBC regarding the wall-to-diaphragm connections introduced for PC1 *buildings*. According to Mitchell *et al.* (1995), about 300 tilt-up structures were damaged in the 1994 Northridge earthquake, mainly due to the lack of proper connection between the roof diaphragm and the tilt-up wall panels. In addition, 1997 UBC provisions concerning design of precast concrete structures in regions of high seismicity were adopted into the 1997 edition of the NEHRP Provisions (Ghosh, 2001a).

According to the Cement Association of Canada (CAC, 2003), PCW *buildings* have been constructed in Canada for over 30 years. However, there have not been any building code requirements in Canada for PCW buildings until very recently (Adebar *et al.*, 2014). CSA A23.3-94 introduced new seismic design requirements for tilt-up wall panels by means of required tension ties in the transverse and longitudinal directions and around the perimeter of the structure to "effectively tie the elements together." In addition, a minimum factored tie force of 5 kN/m is required for top and bottom panel connections.

While CSA A23.3-94 provided comprehensive guidelines for dealing with vertical and transverse out-of-plane loading, the connection requirements required further development. In 2003, the Cement Association of Canada (CAC, 2003) published a design standard guide as an aid for designing tilt-up concrete wall panels. The CAC intended to present information in a manner that serves as an extension to CSA A23.3. According to the CAC guideline (2003), design requirements for connections to tilt-up panels generally followed traditional methods set out in CSA A23.3-94. The connections should be designed to resist forces equal to or greater than the maximum load imposed on the panel component.

The CAC guideline recommendations were later introduced in CSA A23.3-04, with major revisions to the seismic requirements for tilt-up structures stipulated in CSA A23.3-94. These revisions were later referenced in the NBC 2005. The main focus of the standard in CSA A23.3-04 was the gravity design of concrete wall panels. In addition, tension ties were required for structural integrity purposes, in both transverse and longitudinal directions and around the perimeter of the structure to effectively tie the elements together. It is well-known that the seismic performance of PCW *buildings* depends not only on the performance of the concrete wall panels but also on the performance of the connection between the wall panels and the roof and floor diaphragms (Devine *et al*, 2008). The panels were also required to have top and bottom connections to resist a minimum factored force of 5 kN/m, perpendicular to the panel. For tilt-up

panels used as shear walls, the connections between panels were required to be designed to provide nominal ductility. It is noteworthy that the requirements mentioned above are general requirements. While CSA A23.3-04 specifies seismic requirements for tilt-up wall panels in moderately ductile moment resisting frames and squat shear walls, no specific seismic detailing is provided for PCW *buildings*.

The most recent edition of CSA A23.3-14 features separate sections specifying the seismic design provisions for precast wall panels and tilt-up wall panels; it also includes the seismic design provisions for tilt-up wall panel connections to the floor and roof diaphragms. The structural integrity requirements in CSA A23.3-14 are similar to its earlier 2004 edition, except that it specifies a minimum thickness of 3 mm for steel ties, and includes a statement that the structural integrity connection is for providing a load path to the SFRS. Regarding the seismic design requirements for horizontal top and bottom connections of tilt-up panels, CSA A23.3-14 is similar to its earlier 2004 edition, except that the connections are required to resist the factored force of 5 kN/m or 2% of the total factored vertical load for which the wall panel is designed, whichever is greater. It is also specifically stated that the preceding requirements are for the connection of wall panels to roof and floor diaphragms.

Based on the preceding discussion, NBC 2015 is chosen as the *benchmark NBC edition* for PCW buildings.

A.3.2.5 Precast concrete frame (PCF)

Precast concrete frame (PCF) is essentially a post-and-beam system in concrete where columns, beams, and slabs are prefabricated and assembled on site. Concrete shear walls may be present. Various types of members are used. Vertical load-carrying elements may be Ts, cross shapes, or arches and are often more than one storey in height. Beams are often Ts and double Ts, or rectangular sections.

No benchmark year is specified for PC2 (equivalent to PCF) building type in any of the three editions of FEMA P-154; however, some credit is given in the process of quantitative structural scoring, which is in conformity with ASCE 7-16 where the ductility capacity of PC2 building type is acknowledged. Note that ASCE 7-16 permits the use of intermediate PC2 for all SDCs, but it does not allow the use of ordinary PCW for SDC C and higher. NIST Standards specify the benchmark year of 2000 for this building type if designed based on IBC and FEMA 178, FEMA 356/ASCE 41, respectively. The benchmark year of 1998 and 1992 is specified by NIST Standards for this building type if designed based FEMA 310/ASCE 31 and FEMA 178, respectively.

According to Mitchell *et al.* (1995), although specific seismic design requirements for precast concrete structures were not given in Canadian codes at the time of their report publication in 1995, the CPCI Structural Design Manual (CPCI, 1987) provided guidelines for the design of a nominally ductile one-storey precast building. Following the 1994 Northridge earthquake,

Mitchell *et al.* (1995) reported some evidence of column failure, inadequate connection of the diaphragm to the SFRS, and loss of support and failure of spandrel beams in some precast concrete frame *buildings*, yet with no damage in the shear walls.

Until recently, Canadian codes have not specifically addressed the design issues in PCF buildings. CSA A23.3-94 introduced new seismic design requirements for precast elements by requiring tensile ties in the transverse and longitudinal directions to provide load path to the lateral load resisting system. In addition, a minimum factored tie force of 5 kN/m was required for connections between the precast elements and floor or roof diaphragm.

CSA A23.3-94 did not include the precast concrete moment resisting frame system directly in its specific Section 21, "Special Provisions for Seismic Design"; instead, general design requirements for precast concrete are specified in Section 16. However, in the subsequent 2004 and 2014 editions of CSA A23.3, a specific subsection in Section 21, "Special Provisions for Seismic Design" was assigned to precast concrete, with specific seismic design requirements for ductile moment resisting frames constructed using precast concrete with both ductile and strong connections. The ductility and over-strength factors for this building type are taken as the same as those for CMF with cast-in-place concrete in CSA A23.3-04 and CSA A23.3-14. The ductility and over-strength factor were considered for this building type in CSA A23.3-04 and CSA A23.3-14 (referenced in the NBC 2005, 2010, and 2015).

Based on the preceding discussion, NBC 2005 is chosen as the *benchmark NBC edition* for PCF buildings.

A.3.3 **Benchmark NBC editions for steel buildings**

The seismic design of steel *buildings* in Canada is done in accordance with NBC and CSA S16 requirements. CSA S16 is intended for use in designing steel structures, and provides requirements pertaining to the design, fabrication, and erection of steel structures and the structural steel components framed in other building materials. CSA S16 is the steel design standard analogous to the document published by the ASCE in the U.S., which defines the design and detailing rules for standard SFRS. The eighth and most recent edition of CSA S16 was published in 2014. The determination of *benchmark NBC edition* for different types of steel *buildings* is discussed in detail, hereunder.

A.3.3.1 Steel moment frame (SMF)

Steel moment frame (SMF) buildings have a frame of steel columns and beams. In some cases, the beam-column connections have very low moment-resisting capacity; in other cases, some of the beams and columns are fully developed as moment frames to resist lateral forces. The structure is usually concealed on the outside by exterior walls, which can be of almost any material – curtain walls, brick masonry, or precast-concrete panels – and on the inside, covered by ceilings and column furring.

In the first edition of FEMA P-154, 1976 was specified as the benchmark year for S1 (equivalent to SMF) buildings. However, the benchmark year shifted to 1994 in the second and third editions of FEMA P-154. The reason is that seismic requirements for S1 *buildings* were significantly improved in the 1994 UBC, based on the lessons learned from the performance of beam-column connections during the 1994 Northridge earthquake. After the Northridge earthquake, it was observed that most of the post-1980 special moment resisting frames, equivalent to a ductile moment resisting frame SFRS in CSA S16, suffered fractures at beam-column joints (Tremblay *et al.*, 1995). Furthermore, the extent of S1 *building damage* in Northridge did not appear to have been limited to a given range of building height (Ross and Mahin, 1994). The causes of the observed deficiencies in the beam-column connections in S1 buildings were promptly identified, and the state-of-practice was revised to ensure that building structures can survive much longer during earthquakes (Tremblay *et al.*, 1995).

The implications of the 1994 Northridge earthquake on Canadian steel construction were significant. The major revisions in the 1994 UBC are analogous to the major revisions introduced in CSA S16-01, which is referenced in the NBC 2005 (Tremblay *et al.*, 2010), and include provisions for three categories of moment-resisting frames: Type D (ductile), Type MD (moderately ductile), and Type LD (limited ductility) moment-resisting frames.

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for SMF buildings.

A.3.3.2 Steel braced frame (SBF)

Unlike SMF *buildings* that rely on the moment capacity of beam-column connections to provide lateral resistance, steel braced frame (SBF) *buildings* are braced with diagonal members to provide a continuous load path for earthquake loads. Braced frames develop resistance to seismic forces by the bracing action of the diagonal members. Three variations in the configuration and design of braced frames exist:

- Concentrically braced frames: Component work lines intersect at a single point or at multiple points such that the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component connected at the joint.
- Eccentrically braced frames: Component work lines do not intersect at a single point, and the distance between the intersecting work lines (or eccentricity) exceeds the width of the smallest component connecting at the joint.
- Buckling-restrained braced frames: Special types of concentrically braced frames where the steel bracing members are encased within a rigid casing that is intended to prevent buckling of the steel brace. It is noteworthy that this type of braced frame was first introduced in the NBC 2010.

In the first edition of FEMA P-154, 1988 was specified as the benchmark year for SBF *buildings*. In the second and third editions of FEMA P-154, the benchmark year shifted to 1997. Major improvements in seismic requirements for SBF buildings were introduced in the AISC Seismic Provisions for Structural Steel *Buildings* (AISC, 1997) following observations made of steel braced frame performance after the 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes. Although a substantial number of concentrically braced frames experienced brace buckling during these three seismic events (Sabelli *et al.*, 2013), no braced frame structures collapsed or appeared to be at an incipient collapse (EERI, 1996). Tremblay *et al.* (1995) observed numerous occurrences of severe brace overall buckling and tension yielding during their reconnaissance visits, which highlights the importance of properly detailing bracing members in concentrically braced frames that require ductile behavior for earthquake resistance. Tremblay *et al.*'s observations also revealed the potential consequences that inelastic buckling of braces could have on non-structural components such as cladding, partitions, or mechanical equipment in *buildings*, which could be highly hazardous to life safety or impact the post-earthquake functionality of critical facilities (e.g., hospitals). Furthermore, numerous failures occurred in members other than the bracing elements along the seismic lateral load path of concentrically braced frames, such as cracks in floor diaphragms, failures of brace connections, torsional deformations of beams, and tensile fracture of anchor bolts or base plates (Tremblay *et al.*, 1995). Recent seismic design provisions for SBF have implemented a capacity design philosophy to ensure acceptable performance of the entire lateral load path.

Based on the extensive research and studies that followed the 1994 Northridge earthquake, major revisions were introduced in the seismic provisions of CSA S16-01, which was later referenced in the NBC 2005 (Mitchell *et al.*, 2003). CSA S16 required connections of steel braced frames to: 1) have a ductile failure mode or be designed for increased seismic loads in steel systems used in conventional construction in high seismic regions, and 2) satisfy specific minimum capacity design and detailing provisions for systems other than conventional construction (i.e., limited, moderate, or full ductile braced frames). In addition, the yield strength in ductile elements was limited to ensure a minimum level of plastic deformation; requirements were also given to reduce the risk of brittle failures in thick plates, heavy shapes, and welds.

Buckling-restrained braced frames (BRBFs) were introduced in Canada at the end of 1990s. Since then, their use has been expanded considerably, especially in high seismicity regions along the west coast of Canada (Tremblay et al., 2016). Seismic provisions for BRBFs were first adopted in CSA S16-09, which was referenced in the NBC 2010.

Based on the preceding discussion, the NBC 2010 is chosen as the *benchmark NBC edition* for buckling-restrained braced frames and the NBC 2005 is chosen as the *benchmark NBC edition* for other types of braced frames.

A.3.3.3 Steel light frame (SLF)

Steel light frames (SLF) *buildings* are typically agricultural structures, industrial factories, and warehouses. They are usually one storey in height, sometimes without interior columns, and often enclose a large floor area. Construction is typically of steel frames that span the shortest building dimension and resist lateral forces as moment frames. Forces in the long direction are usually resisted by diagonal steel-rod bracing. These buildings are usually clad with lightweight siding. This type of building has performed relatively well in past earthquakes and collapses do not typically occur.

No benchmark year is specified for S3 buildings (equivalent to SLF) in any of the three FEMA P-154 editions. However, in the third edition of FEMA P-154, post-benchmark score modifiers are provided for this building type. It is noted that the ductility capacity of S3 buildings is acknowledged in ASCE 7-16.

Since SLF *buildings* generally consist of moment frames in the transverse direction and diagonal steel-rod braced frame in the longitudinal direction, from a seismic resistance point of view, their seismic performance is similar in the transverse direction to SMF systems, and in the longitudinal direction to SBF systems.

Given that major improvements were introduced in the seismic provisions of CSA S16-01, which was later referenced in the NBC 2005, the *benchmark NBC edition* of SLF buildings is chosen as the NBC 2005.

A.3.3.4 Steel frame with concrete shear walls (SCW)

The construction of steel frames with concrete shear walls (SCW) buildings is similar to that of SMF buildings in that a matrix of steel columns and girders is distributed throughout the structure. The joints, however, are not designed for moment resistance, and the lateral forces are resisted by concrete shear walls. The shear walls may be part of the elevator/service core, exterior walls or interior walls.

In the first and second editions of FEMA P-154, 1976 was specified as the benchmark year for S4 (equivalent to SCW) buildings. In the third edition of FEMA P-154, 1994 was specified as the benchmark year for this building type.

The structural configuration of SCW buildings is similar to that of CSW buildings (with the exception of different required seismic detailing for the connection of the concrete shear walls to the steel roof and floor). Given the structural similarities and providing that CSW buildings have performed well during previous earthquakes, SCW buildings are expected to exhibit good seismic performance.

Since major improvements were made in the seismic provisions of CSA A23.3-04, which was later referenced in the NBC 2005, the NBC 2005 is chosen as the *benchmark NBC edition* for SCW buildings.

A.3.3.5 Steel frame with infill masonry shear walls (SIW)

Steel frames with infill masonry shear walls (SIW) are part of an older type of building construction that consists of steel structural frames infilled with unreinforced masonry. In older buildings, the diaphragms are often wood. More recent buildings have reinforced concrete floors. Because of the masonry infill, the structure tends to be very stiff. In major earthquakes, the infill walls may suffer substantial cracking and deterioration, thus reducing their stiffness. This, in turn, puts additional demands on the frame. Some of the walls may fail while others remain intact, which may result in torsion or soft-storey problems.

No benchmark year is specified for S5 buildings (equivalent to SIW) in any of the three FEMA P-154 editions. This is attributed to the fact that S5 buildings are not expected to have any appreciable level of ductility, as opposed to requirements of modern seismic design standards, in accordance with ASCE 7-16 in which the ductility capacity of S5 buildings is not acknowledged. Due to inferior seismic performance of S5 buildings, ASCE 7-16 restricts the use of this building type in seismic design categories A and B. The NIST Standards specify a benchmark year of 2000 for SIW *buildings* if they are designed based on the IBC, FEMA 178, or FEMA 356/ASCE 41, or 1998 for SIW *buildings* if they are designed based on FEMA 310/ASCE 31.

In the NBC 2005 and its successive editions, SIW is identified as other steel SFRS, and no ductility capacity is acknowledged. Since the seismic performance of SIW *buildings* is expected to be similar to CIW *buildings* (NRC, 1993b), the NBC 2005 is chosen as the *benchmark NBC edition* for SIW buildings.

A.3.4 **Benchmark NBC editions for masonry buildings**

The seismic design of masonry *buildings* in Canada is conducted in accordance with the requirements of the NBC and CSA S304 (CSA, 2014). CSA S304 is intended for use in designing masonry structures and specifies requirements for the structural design of unreinforced and reinforced masonry structures and components. The current edition of CSA S304 was published in 2014. In the following sections, the determination of benchmark years for different types of masonry *buildings* is discussed.

A.3.4.1 Reinforced masonry bearing walls with light wood or metal deck diaphragms (RML)

Reinforced masonry bearing walls with light wood or metal deck diaphragms (RML) buildings are mostly low-rise perimeter bearing-wall structures, often with wood diaphragms. Floor and roof assemblies usually consist of timber joists and beams, glue-laminated beams or light steel joists. The bearing walls consist of grouted and reinforced hollow or solid masonry units. Interior supports, if any, are often wood or steel columns, wood-stud frames or masonry walls.

In the first edition of FEMA P-154, 1976 was determined as the benchmark year for RM1 (equivalent to RML) buildings. In the second and third editions of FEMA P-154, the benchmark year shifted to 1997. This is because in UBC 1997, increased strength of the anchorage was required for out of plane connections of concrete and masonry walls to flexible diaphragm (Ghosh, 2001b). The 1997 UBC also defined specific criteria for the analytical definition of rigid versus flexible diaphragms (Shipp and Hart, 2011). Flexible diaphragms experience amplified motion caused by the seismic forces transferred from relatively rigid masonry walls. Therefore, for flexible diaphragms in seismic zones 3 and 4, the 1997 UBC requires the anchorage force values to be increased by 50 percent.

According to Anderson and Brzev (2009), most of the CSA S304.1-04 seismic requirements for shear wall reinforcement existed in the 1994 edition of the standard. In CSA S304.1-95, while some minimum seismic design requirements for reinforced masonry walls (i.e., conventional reinforced masonry shear walls) were included, seismic design requirements for nominal (i.e., moderate) ductile reinforced masonry shear walls were referred to in its Appendix, which was not a mandatory part of CSA S304.1-94. However, in CSA S304.1-04, the seismic design requirements for moderate (i.e., nominal) ductile reinforced masonry shear walls were introduced in the main body of the standard. The NBC 2005 referenced CSA S304.1-04, in which limited and ductile shear walls were introduced for the first time in the main body of the standard. While the static method is the default method of analysis in the NBC 1995, the dynamic method is the default method in the NBC 2005, and the static method is restricted to certain structures and seismic hazards. Structure irregularities are also better defined in the NBC 2005, with more stringent requirements.

The NBC 2005 also requires that diaphragms and their connections be designed so as not to yield. Furthermore, flexible diaphragms and their connections are required to be designed and detailed according to applicable referenced design standards such as CSA O86 for wood diaphragms, and CSA S16 for metal deck diaphragms. The connections between the diaphragm and lateral-load resisting components are extremely important for maintaining structural integrity; for this reason they are required to be designed to remain elastic during seismic loading. The ductility and over-strength factors were also introduced for this building type in the 2004 and subsequent editions of CSA S304 (referenced in the NBC 2005 and its subsequent editions).

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for RML buildings.

A.3.4.2 *Reinforced masonry bearing walls with concrete diaphragms (RMC)*

Reinforced masonry bearing walls with concrete diaphragms (RMC) buildings are similar to RML buildings, except that the diaphragms are made of concrete. Earthquake-related damage is mainly caused by poor anchorage connections in precast concrete diaphragms.

In the first edition of FEMA P-154, 1976 was determined as the benchmark year for RM1 (equivalent to RML) buildings. In the second and third editions of FEMA P-154, the benchmark year shifted to 1994. This is because the minimum strength requirement for anchorage of walls to rigid diaphragms was first introduced in the 1994 edition of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. This is the resource document for seismic design provisions in all model building codes in the U.S. (except the 1997 UBC, which has its own design provisions) (Ghosh, 2002). The document underwent significant changes from its 1991 to 1994 and 1997 editions. Over the years it has become more like a code or standard. It was stated earlier that improperly connecting the precast concrete diaphragm to the wall can lead to significant damage during an earthquake event.

In addition to the prescribed seismic requirements for RMC in the different edition of CSA S304.1 and the NBC, the NBC 2005 and its subsequent editions also require a torsional sensitivity analysis of *buildings* with rigid diaphragms, including RMC buildings.

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for RMC buildings.

A.3.4.3 Unreinforced masonry bearing wall buildings (URM)

Unreinforced masonry bearing wall buildings (URM) buildings have perimeter bearing walls that consist of unreinforced clay brick, stone, or concrete masonry. Interior bearing walls, where present, also consist of unreinforced clay brick, stone, or concrete masonry. Construction varies according to type of use, although wood floor and roof diaphragms are common in older buildings. URM buildings are recognized as perhaps the most hazardous type of SFRS. They have been observed to fail in many modes during previous earthquakes.

In the first edition of FEMA 154, no benchmark year was specified for URM buildings. In its second edition, FEMA 154 specified the benchmark year as 1991 (though no post-benchmark score modifier is provided for this building type). In its third edition in 2015, similar to its first edition, no benchmark date is defined by FEMA P-154. This can be attributed to the fact that URM *buildings* are not expected to have appreciable ductile behaviour, in accordance with ASCE 7-16, in which the ductility capacity for URM buildings is not acknowledged. Due to the inferior seismic performance of URM buildings, ASCE 7-16 does not permit the use of URM *buildings* in SDC C or higher. The NIST Standards specify the benchmark year of 1991 for this building type, if designed based on the UBC.

According to FEMA 774 (FEMA, 2009), the Los Angeles City Council passed an ordinance in 1981 requiring structural upgrading or demolition of 14,000 unreinforced masonry *buildings*, except for residential *buildings* that had four or fewer dwelling units. The 1985 Mexico City earthquake that caused over 10,000 deaths motivated the Los Angeles City Council to accelerate the time table for compliance.

Reports from past earthquakes have also shown that unreinforced masonry suffers extensive damage in earthquakes (e.g., 1994 Northridge earthquake). A basic visual assessment of masonry *buildings* after the 1994 Northridge earthquake, performed on about 140 structures by a team from The Masonry Society (TMS), demonstrated considerable progress since the San Fernando earthquake of 1971. According to TMS (1994), modern reinforced masonry generally performed quite well, while, as expected, many older unretrofitted unreinforced masonry structures were badly damaged (including complete collapse, collapsed walls, fallen parapets, or severe diagonal cracking of piers). Nevertheless, older unreinforced masonry structures that had been retrofitted to comply with Los Angeles' Division 88 requirements – which required that all URM parapets be removed or braced, mortar joints be tested and strengthened to meet minimum values for shear strength, and all URM walls be anchored to floor and roof diaphragms – performed much better.

The ban on unreinforced masonry *buildings* was first introduced in the NBC 1975, which explicitly prohibited the construction of unreinforced masonry *buildings* in moderate to severe seismic regions. This was also enforced in the NBC 1990 and 1995 for seismic velocity zone 2 or higher, but exempted walls 200 kg/m^2 or less that were under three metres tall. The NBC 1995 banned unreinforced masonry SFRS for *buildings* with more than three storeys in seismic zone 2 or higher. This ban potentially implied that unreinforced masonry SFRS in seismic zone 0 and 1 was permitted with no height restriction, provided that the design met the minimum reinforcement requirements for bearing walls, and that the tensile and compressive stresses were below permitted values (Anderson and Brzev, 2009). Part 9 of NBC 1995 allowed URM *buildings* for small housing construction in zone 2 or higher, with some restrictions on maximum permitted overall building height (i.e., maximum of one storey in zone 4, and maximum of three storeys in zones 2 and 3).

CSA S304.1-04 and CSA S304-14 only allow the use of unreinforced masonry at sites where $I_E F_a S_a(0.2) < 0.35$ g, but exempt masonry partition walls 200 kg/m^2 or less that are under three metres tall, and located at sites where $I_E F_a S_a(0.2) < 0.75$ g.

According to CSA S304.1-04 and CSA S304.1-14, URM buildings are not permitted where the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$ g. In addition, the NBC 2005 and its later editions set restrictions on maximum permitted overall building height at sites where $I_E F_a S_a(0.2) < 0.35$ g for different SFRS(s), and required seismic analysis (i.e., dynamic and equivalent static) methods (DeVall, 2003; Varadharajan *et al.,* 2013). The unreinforced masonry walls are introduced in Table 4.1.8.9 of the NBC 2005 and its later edition, for which a maximum overall height of 15 m and 30 m is set when 0.2 g $\leq I_E F_a S_a(0.2) < 0.35$ g and $I_E F_a S_a(0.2) < 0.2$ g, respectively.

Based on the preceding discussion, the NBC 2005 is chosen as the benchmark NBC edition for URM buildings.

A.3.5 **Benchmark NBC edition for other building types**

This section introduces two additional building types, namely cold-formed steel (CFS) buildings and manufactured homes, which do not fall into any of the previous four building categories (i.e. wood, steel, concrete, and masonry).

A.3.5.1 Cold-formed steel buildings (CFS)

No benchmark year is specified for colded-formed steel (CFS) buildings in any of the three FEMA P-154 editions. Remember that the building types in the third edition of FEMA P-154 are based on the common building types listed in ASCE 41-13. However, the CFS building type has not been considered as a common building type until ASCE 41-17.

The seismic design of CFS buildings is in accordance with the requirements of the NBC and CSA S136. CSA S136 is intended for use in designing cold-formed steel structural members and provides well-defined procedures for designing load-carrying cold-formed steel members in *buildings* and other applications, provided that proper allowances are made for dynamic effects. The ninth edition of CSA S136 was published in 2016.

The CSA S136-07 and CSA S136-12 are referenced in the NBC 2010 and 2015, respectively, to include seismic design provisions as well as ductility and over-strength modification factors for cold-formed steel SFRSs to resist wind and seismic forces in a wide range of buildings constructed with cold-formed steel framing.

Since the cold-formed steel members were first recognized as SFRS in the NBC 2010 with specification of corresponding ductility and over-strength modification factors, the NBC 2010 is chosen as the *benchmark NBC edition* for CFS buildings.

A.3.5.2 Manufactured homes (MH)

There are two types of manufactures homes (MH): mobile homes and modular buildings. Manufactured homes in Canada are designed and installed in accordance with the requirements of the NBC and CSA Z240.10.1, which is referenced by the NBC. CSA Z240.10.1 is intended to be used by installers, inspectors, and owners of manufactured homes designed to be supported on longitudinal floor beams. The current edition of CSA Z240.10.1 was published in 2016.

The seismic design of manufactured homes in Canada follows the requirements of Part 9 of the NBC, where it stipulates that building frames are required to be anchored to the foundation unless structural analysis of wind and earthquake forces indicate that anchorage is not needed. It should be recalled that the *Level 1 – PST* can only be used to screen existing buildings covered by Part 4 of the NBC..

Mobile homes are prefabricated, permanently attached to a chassis, and can be relocated either by being towed or transported on a trailer. They are typically 4.3 meters wide, not more than one storey in height, and are raised above ground by concrete blocks or metal supports, but without

being anchored to the ground. Numerous studies showed that mobile homes performed significantly worse than conventional wood-frame dwellings in the 1994 Northridge earthquake (SSC, 1995). This is due to the fact that, when exposed to seismic actions, a mobile home's main weakness is its foundation. Californian state regulations for mobile home installations were notoriously weak prior to the 1994 Northridge earthquake (SSC, 1995). Since a mobile home is generally placed on jacks or other supports without proper consideration of the structure's resistance to wind or seismic forces, the most common failure is a mobile home's shifting from its foundation during an earthquake, which could be prevented with earthquake-resistant bracing (ERB) systems. Unfortunately, these bracing systems were installed in a very small number of mobile homes prior to 1994. Based on the observed performance of mobile homes during the 1994 Northridge earthquake, new requirements for mobile home construction appeared in several California building regulations in and around 1995. The new requirements included installing an approved ERB system and providing a flexible gas connection fitted with an automatic shutoff device. It should be noted that the three main U.S. codes (i.e., the NBC, UBC, and IBC) do not apply to manufactured homes in California (FEMA P-154, 2015). Thus, in the third edition of FEMA P-154, the benchmark year was set as 1995 for manufactured homes (i.e., MH building type) for California. This building type did not exist in the first and second editions. For other states, the benchmark year was set as 2009 for the MH building type, since the U.S. Department of Housing and Urban Development's Installation Standard required tie-downs in this type of construction after October 2008 (FEMA P-154).

In the NBC 1995, anchorage of small housing (not more than 4.3 m wide and not more than one storey in height) to the foundation in accordance with the requirements of CSA Z240.10.1-94 was optional. However, beginning in 2005, the NBC has explicitly required that small housing *buildings* be anchored to their foundations, in conformance with the requirements of CSA Z240.10.1-94. Given the limit set by CSA Z240.10.1 regarding the size of manufactured homes (less than 4.3 m in width and less than one storey in height), these *buildings* are expected to be light, and therefore are not expected to experience large seismic forces. This might justify the fact that only wind is taken into consideration in anchorage design in CSA Z240.10.1 (i.e., wind governing over seismic forces). Examining Clause 9.23.6.1 of the NBC 2015, it is evident that anchor bolt size and spacing values are independent of all seismic hazard values for one-storey buildings, which indirectly implies that wind pressures govern lateral load design over seismic forces.

Modular *buildings* are factory built in units or modules, but do not have permanent chassis or axles and must be transported to the site on flatbed trucks. The modules are set in place using a crane, and then are joined together. Modular *buildings* are typically placed on traditional permanent concrete foundations, with or without basements. For large manufactured structures more than 4.3 m wide and an overall height not exceeding three storeys, the 1995 and later editions of the NBC require such building frames to be anchored to their foundations unless a structural analysis of wind and earthquake pressures shows anchorage is not required. Furthermore, the NBC 2010

and 2015 impose more stringent seismic anchorage requirements for MH *buildings* more than 4.3 m wide and an overall height not exceeding three storeys, consisting of tighter spacing and larger anchor sizes (depending on the seismicity of the site on which the MH is built). Note that prefabricated structures that are permanently anchored to a foundation are not considered to be the MH building type. In such cases, the appropriate building type should be chosen based on the SFRS used in the structure.

Based on the preceding discussion, the NBC 2005 is chosen as the *benchmark NBC edition* for MH buildings with width not greater than 4.3 m and building height not greater than one storey. The NBC 2015 is chosen as the *benchmark NBC edition* for MH buildings with width greater than 4.3 m and building height not greater than three storeys.

This page is intentionally left blank.

APPENDIX B CHARACTERISTICS AND EARTHQUAKE PERFORMANCE OF MODEL BUILDING TYPES IN THE LEVEL 1 – PST

This Appendix outlines the characteristics and typical *damage* of seventeen *model building types* considered in the *Level 1 – PST*.

B.1 Wood buildings

B.1.1 Wood light frame (WLF)

Engineered Wood light frame (WLF) *buildings* (Figure B.1) are typically apartment, commercial, or office *buildings* of up to 6 storeys, or having a *building* area exceeding 600 m^2. WLF buildings are covered by Part 4 of the NBC and designed in accordance with the CSA O86 Engineering Design in Wood.

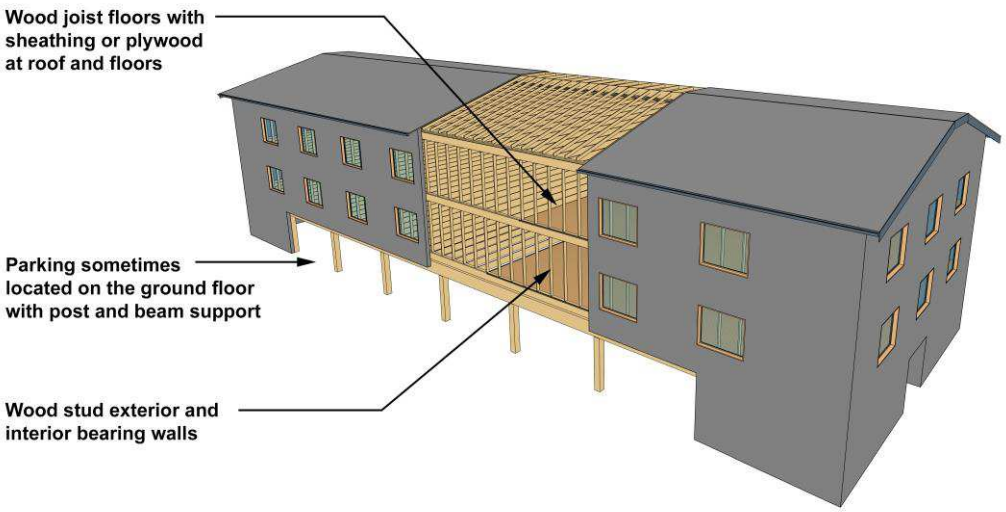


Figure B.1: Engineered wood light frame (WLF) building (reproduced from FEMA 547, FEMA, 2006)

Wood stud walls are typically constructed of nominal 2-inch by 4-inch (38 × 89 mm) vertical wood members spaced at 16 inches (400 mm) on centre. Studs of nominal size 2-inch by 6-inch (38 mm × 140 mm actual size) are typically used for multiple storeys, or to meet energy requirements.

The shear wall construction of WLF is similar to that of non-engineered WLF covered by Part 9 of the NBC with an exception being that shear walls of WLF have hold-downs at the ends of each shear wall segment or shear line. Special seismic design provisions are included, especially for high seismic areas, including considerations for capacity-based design of the floor and roof diaphragms, drag struts, chords, and hold-downs.

Older WLF buildings (usually pre-1940) have either no foundations or weak foundations constructed of unreinforced masonry or poorly reinforced concrete.

B.1.2 **Wood post-and-beam (WPB)**

Engineered wood post-and-beam (WPB) buildings are typically commercial and industrial *buildings* which are covered by Part 4 of the NBC (Figure B.2). These are generally larger *buildings*, such as warehouses, offices, churches, theatres, fire stations, or even large gas stations. Post-and-beam timber construction consists of larger rectangular timber columns (140 mm × 140 mm or larger) or sometimes circular timber columns framed together with large wood beams or trusses. The structure is sometimes encased by different types of external walls, including masonry and stone veneers.

Figure B.2: Engineered wood post-and-beam (WPB) building (reproduced from FEMA 547, FEMA, 2006)

Traditional timber-frame construction is characterized by traditional wooden joineries such as Mortise and Tenon joined with wood pegs or wedges. The joints rely mostly on using the wood in compression and they are detailed in a way that undesirable failure modes, such as tension perpendicular to grain and shear, are avoided. The other seismically important elements in a traditional timber frame are knee braces. Their primary function is to provide rigidity and stiffening of the connection where major beams and columns join, providing racking resistance to the frame.

Modern post-and-beam construction typically uses mechanical fasteners, such as steel plate connectors, to join the structural members together. This type of construction usually uses diagonal members to provide adequate lateral bracing, while maintaining open space. In many cases the steel is inserted within the timber members as a hidden knife plate to provide an appearance of traditional joinery.

Well-designed timber frame buildings may not need to rely on shear walls or infill framing to achieve lateral stability. However, it is generally recommended that structural loads on timber frames be limited to gravity loads and that lateral loads be carried by other lateral load bracing systems, such as conventional shear walls.

If a WPB building has substantial light frame wood shear walls, it should be classified as a WLF building, since the shear walls are the main seismic force resisting system.

B.2 Steel buildings

B.2.1 **Steel frame buildings**

B.2.1.1 Steel moment frame (SMF)

This type has a frame of steel columns and beams (Figure B.3). In some cases, the beam-column connections have very low moment-resisting capacity; in other cases, some of the beams and columns are fully developed as moment frames to resist lateral forces. The structure is usually concealed on the outside by exterior walls, which can be of almost any material – curtain walls, brick masonry, or precast-concrete panels – and on the inside, covered by ceilings and column furring.

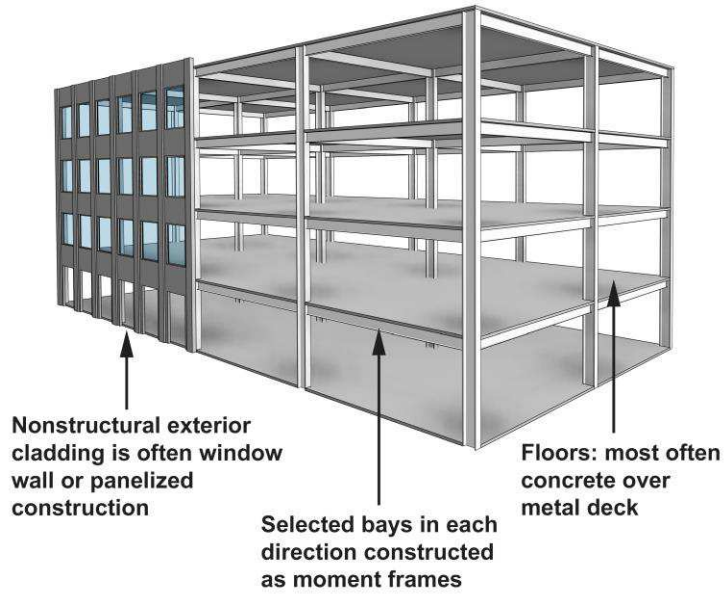


Figure B.3: Steel moment frame (SMF) building (reproduced from FEMA 547, FEMA, 2006)

Typical steel moment-resisting frame structures usually have similar bay widths in both the transverse and longitudinal direction, around 7 to 10 m. The load-bearing frame consists of beams and columns distributed throughout the building. The floor diaphragms are usually concrete, sometimes over steel decking. Moment-resisting frame structures built after 1950 tend to incorporate prefabricated panels hung on the structural frame as the exterior finish. These panels

may be precast concrete, stone or masonry veneer, metal, glass, or plastic. This type of structure is used for commercial, institutional, and other public *buildings*. It is seldom used for low-rise residential *buildings*.

Old steel-frame structures are usually clad or infilled with unreinforced masonry, such as bricks, hollow clay tiles, and terracotta tiles (see type SIW for a detailed discussion). Other frame *buildings* from this period are encased in concrete. Both wood and concrete floor or roof diaphragms are common in these older *buildings*.

B.2.1.2 Steel braced frame (SBF)

Steel braced frame (SBF) structures (Figure B.4) have been built since the late 1800s with similar usage and exterior finish as steel moment frame *buildings*. Braced frames are sometimes used for long and narrow *buildings* because of their stiffness. Although these *buildings* are braced with diagonal members, the bracing members usually cannot be detected from the building exterior.

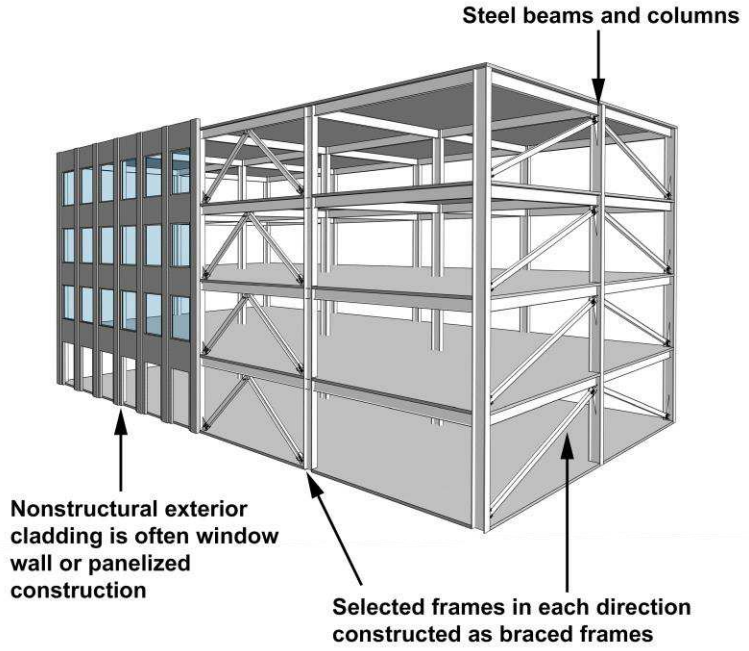


Figure B.4: Steel braced frame (SBF) building (reproduced from FEMA 547, FEMA, 2006)

In concentrically braced frames, the lateral forces or loads are resisted by the tensile and compressive strength of the bracing (see Figure B.4). A recent development in seismic bracing is the eccentric brace. Here the bracing is slightly offset from the main beam-to-column connection, and the short section of beam is expected to deform significantly under major seismic forces and thereby dissipate a considerable portion of energy.

B.2.1.3 Earthquake performance of steel frame buildings

Because of their strength, flexibility and lightness, steel frame *buildings* tend to be more satisfactory in their earthquake resistance as compared to other structure types. Collapse in

earthquakes has been very rare, although steel frame *buildings* did collapse, for example, in the 1985 Mexico City earthquake. In earthquakes that have occurred in the United States, these *buildings* have performed quite well, and probably will not collapse unless subjected to extremely severe ground shaking. Possible damage includes:

1. Non-structural damage to elements such as interior partitions, equipment and exterior cladding, resulting from excessive deflection in frame structures;
2. Cladding and exterior finish material may fall if inadequately or incorrectly connected;
3. Permanent displacements caused by plastic deformation of structural members; and
4. Pounding with adjacent structures may occur.

B.2.2 **Steel light frame (SLF)**

Most light-frame (SLF) *buildings* that exist today were built after 1950 (Figure B.5). They are used for agricultural structures, industrial factories, and warehouses. They are usually one storey in height, sometimes without interior columns, and often enclose a large floor area. Construction is typically of steel frames that span the shortest building dimension and resist lateral forces as moment frames. Forces in the long direction are usually resisted by diagonal steel-rod bracing. These *buildings* are usually clad with lightweight siding. Light frame *buildings* have traditionally been constructed from hot rolled steel sections. In 2010, the NBC officially included cold-formed steel structures for the first time, designed and detailed in accordance with CAN/CSA S136, as a new type of SFRS.

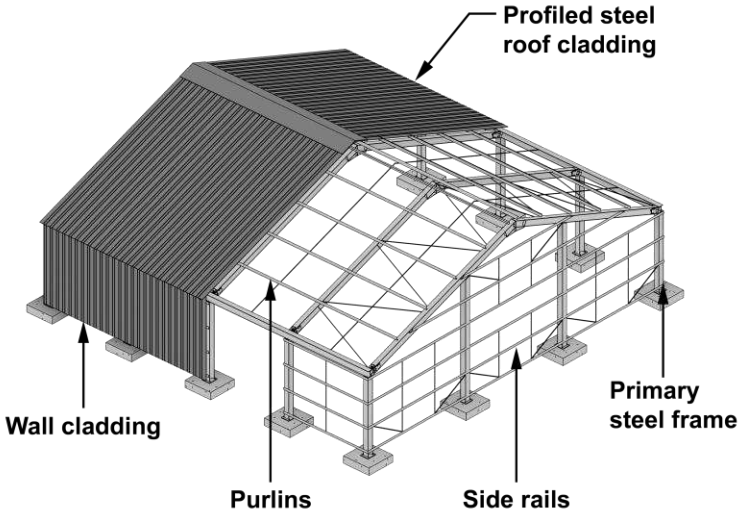


Figure B.5: Steel light frame (SLF) building (reproduced from British Constructional Steelwork Association, Ltd, 2017)

Because these *buildings* are low-rise, lightweight, and constructed of steel members, they usually perform relatively well in earthquakes. Collapses do not usually occur. Some typical problems are as follows:

1. Insufficient capacity of tension braces may lead to their elongation and, in turn, to *building damage;*
2. Inadequate connection to the foundation may allow the building columns to slide; and,
3. Loss of cladding may occur.

B.2.3 Steel frame with concrete shear walls (SCW)

The construction of this structural type (Figure B.6) is similar to that of the steel moment-resisting frame, in that a matrix of steel columns and girders is distributed throughout the structure. The joints, however, are not designed for moment resistance, and the lateral forces are resisted by concrete shear walls.

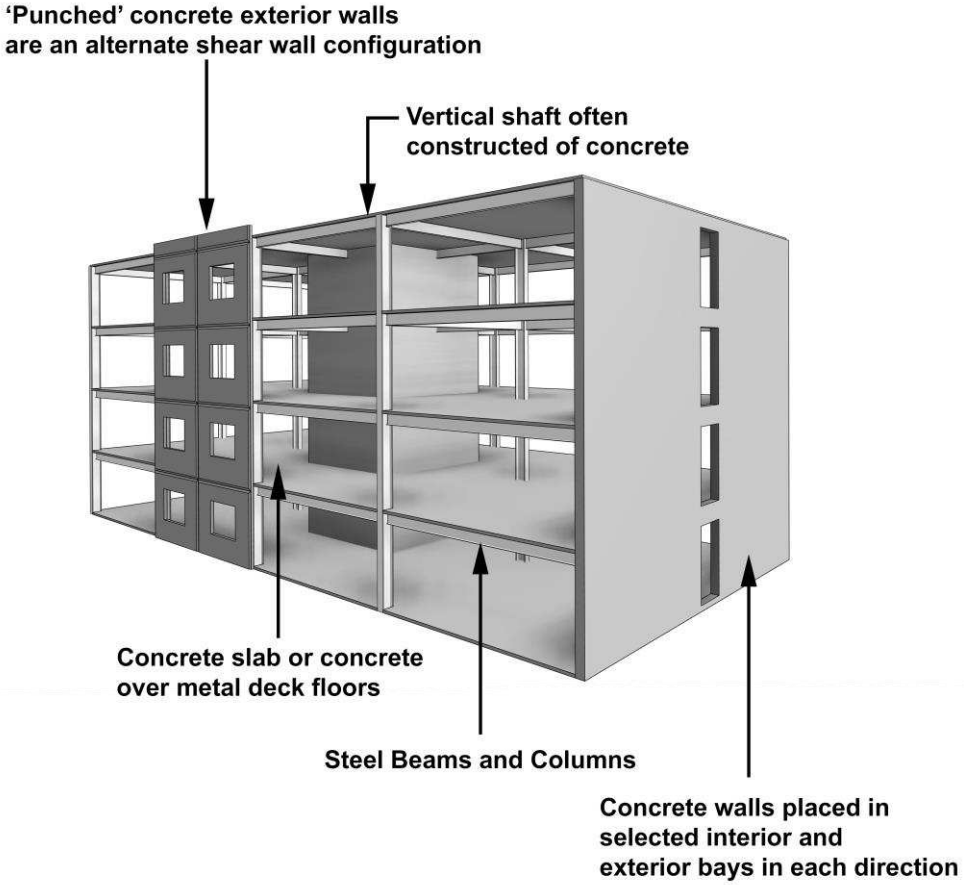


Figure B.6: Steel frame with concrete shear walls (SCW) building (reproduced from FEMA 547, FEMA, 2006)

The shear walls may be part of the elevator/service core, exterior walls or interior walls. This type of structure performs as well in earthquakes as other types of steel *buildings*. Some typical types of damage are as follows:

1. Shear cracking and distress may occur around openings in concrete shear walls;

2. Wall construction joints may be weak planes, resulting in wall shear failure below expected capacity; and
3. Insufficient lap lengths in vertical reinforcing steel may lead to wall bending failures.

B.2.4 Steel frame with infill masonry shear walls (SIW)

Steel frame with infill masonry shear walls (SIW) (Figure B.7) consists of a steel structural frame and walls infilled with unreinforced masonry (URM). In older *buildings*, the diaphragms are often wood. More recent *buildings* have reinforced concrete floors. Because of the masonry infill, the structure tends to be very stiff.

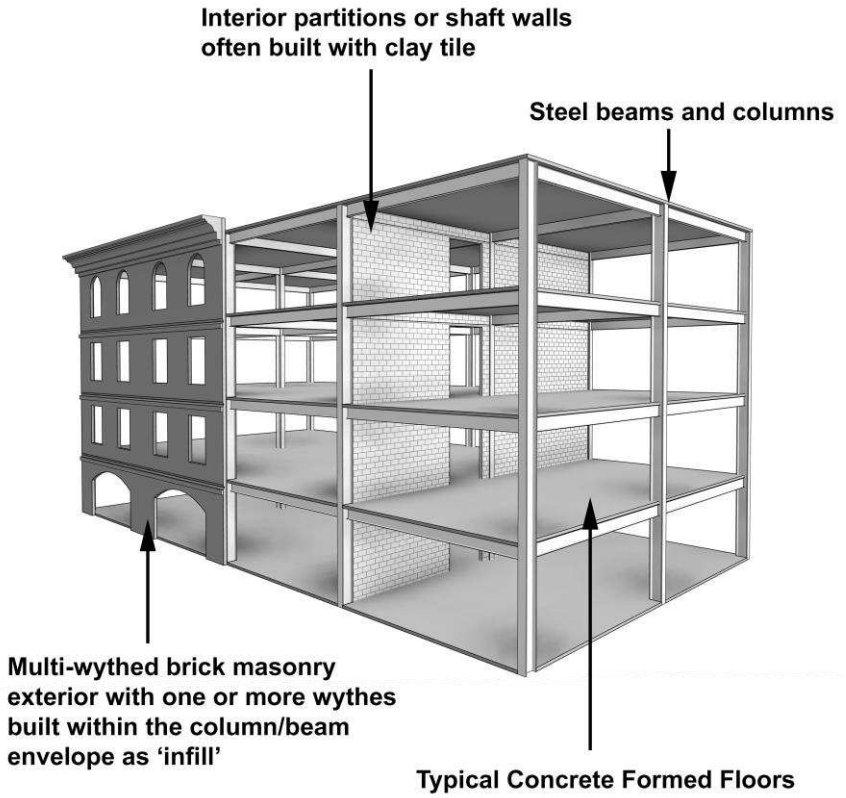


Figure B.7: Steel frame with infill masonry shear walls (SIW) building (reproduced from FEMA 547, FEMA, 2006)

In major earthquakes, the infill walls may suffer substantial cracking and *deterioration*, thus reducing their stiffness. This, in turn, puts additional demands on the frame. Some of the walls may fail while others remain intact, which may result in torsion or soft-storey problems. The hazard from falling masonry is significant, as these *buildings* can be taller than 20 storeys. Typical damage for this building type is as follows:

1. Infill walls tend to buckle and fall out-of-plane when subjected to strong lateral forces. Because the infill walls are non-load-bearing, they tend to be thin (approximately 23 cm maximum) and do not have additional shear strength because of compression from above;

2. Veneer masonry around columns or beams is usually poorly anchored to the structural members and may disengage and fall;
3. Interior infill partitions and other non-structural elements may be severely damaged and collapse;
4. Soft storey, where infill walls exist in the upper storeys but not at the ground floor. The difference in stiffness creates a large demand at the ground floor columns, causing structural damage; and
5. When the earthquake forces are very high, the steel frame itself may fail locally. Connections between members are usually not designed for high lateral loads (except in tall *buildings*) may can lead to damage. Although complete collapse has seldom occurred, it cannot be ruled out.

B.3 Concrete buildings

B.3.1 Concrete moment frame (CMF)

Two construction subtypes fall under concrete moment frame (CMF): (a) non-ductile reinforced-concrete frames without reinforced infill walls, and (b) ductile reinforced-concrete frames. The CMF construction type is illustrated in Figure B.8.

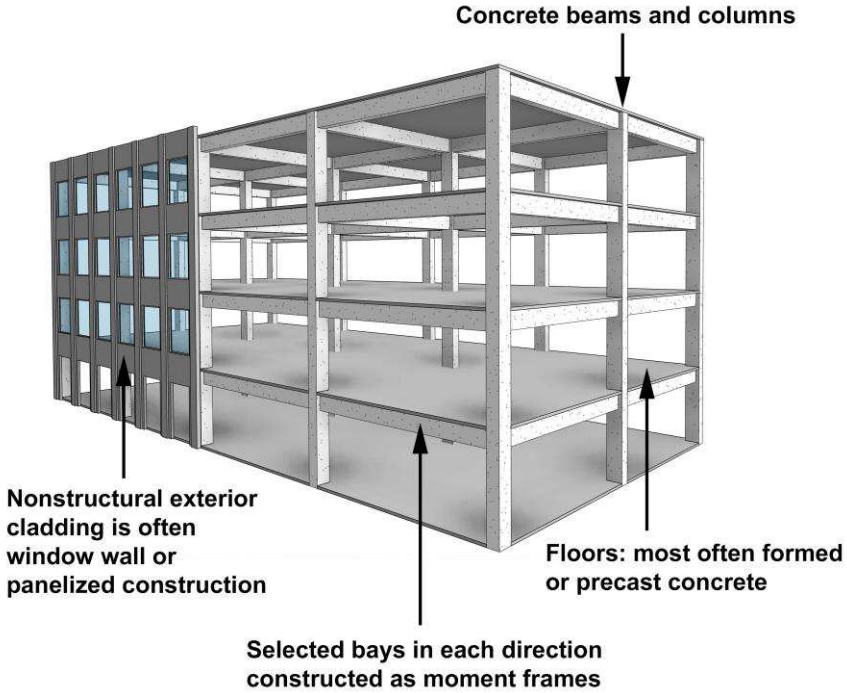


Figure B.8: Concrete moment frame (CMF) building (reproduced from FEMA 547, FEMA, 2006)

The most prevalent CMF buildings consist of non-ductile reinforced concrete frame structures without reinforced infill walls, built between about 1950 and 1972. In many regions of Canada,

this type of construction continues even today. This group includes large multi-storey commercial, institutional, and residential *buildings* constructed using flat-slab frames, waffle-slab frames, and the standard girder-column-type frames. These structures generally are more massive than steel frame *buildings*, are under-reinforced (i.e., have insufficient reinforcing steel embedded in the concrete) and display low ductility. Some typical problems are as follows:

1. Large tie spacing in columns may lead to a lack of concrete confinement or shear failure;
2. Placement of inadequate rebar splices at the same location may lead to column failure;
3. Insufficient shear strength in columns may lead to shear failure prior to the development of moment hinge capacity;
4. Insufficient shear tie anchorage may lead to premature brittle failure in shear or compression;
5. Lack of continuous beam reinforcement may result in hinge formation during load reversal;
6. Inadequate reinforcing of beam-column joints or location of beam bar splices at columns may lead to failures;
7. The relatively low stiffness may lead to substantial non-structural damage; and
8. Pounding damage with adjacent *buildings* may occur.

Recently built concrete moment frames are required to have special reinforcing details to achieve satisfactory ductility. This has been required in Canada's high seismic zones since the mid-1970s.

B.3.2 **Concrete shear walls (CSW)**

This category consists of *buildings* with a concrete shear wall (CSW) structural system or frame structures with CSW (Figure B.9). The entire structure, along with the usual concrete diaphragm, is typically cast-in-place. Such "box" systems were often used in schools, churches and industrial *buildings*. Frame *buildings* with shear walls tend to be commercial and industrial. A common example of the latter type is a warehouse with perimeter concrete walls.

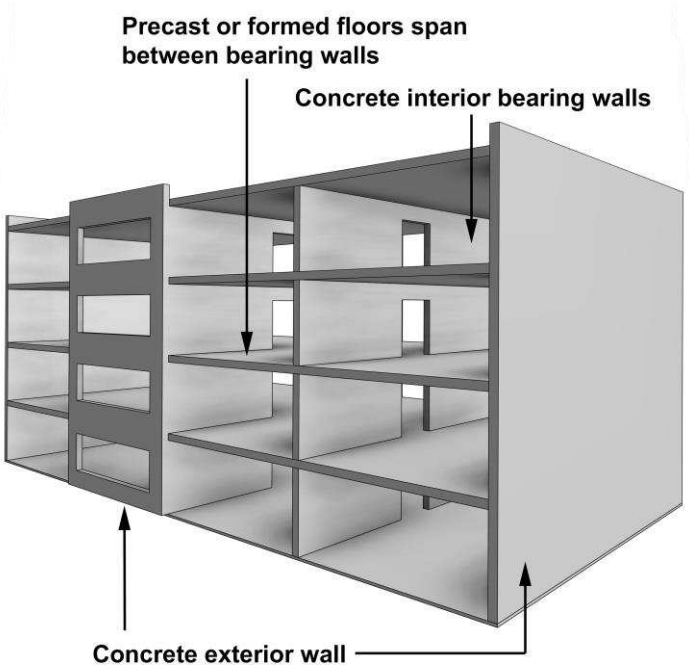


Figure B.9: Concrete shear walls (CSW) building (reproduced from FEMA 547, FEMA, 2006)

Concrete shear wall *buildings* constructed since the early 1950s tend to be institutional, commercial, and residential *buildings*, ranging from one to more than thirty storeys. Residential *buildings* of this type are often mid-rise towers. The shear walls in these newer *buildings* can be located along the perimeter, as interior partitions, or around the service core.

This building type generally tends to perform better than concrete frame *buildings*. They are quite heavy relative to steel frame *buildings*, but they are also stiff due to the presence of shear walls. Some types of damage commonly observed in taller *buildings* are caused by vertical discontinuities, pounding, or irregular configuration. Other damage specific to this building type are as follows:

1. Shear cracking and distress may occur around openings in concrete shear walls during large seismic events;
2. Shear failure may occur at wall construction joints usually at a load level below the expected capacity; and,
3. Bending failures may result from insufficient vertical chord steel and insufficient lap lengths at the ends of the walls.

B.3.3 Concrete frame with infill masonry shear walls (CIW)

This category consists of *buildings* that have a concrete frame with unreinforced-masonry or reinforced-masonry infill walls (Figure B.10). These *buildings* tend to have larger structural members, although the level of reinforcement detailing is doubtful. These concrete frames have

been used for commercial and industrial structures and are generally more than three storeys tall. The hazards of these older *buildings* are similar to and perhaps more severe than those with newer frames. Where unreinforced masonry (URM) infill is present, a falling hazard exists. The failure mechanism for URM infill in a concrete frame is the same as for SIW.

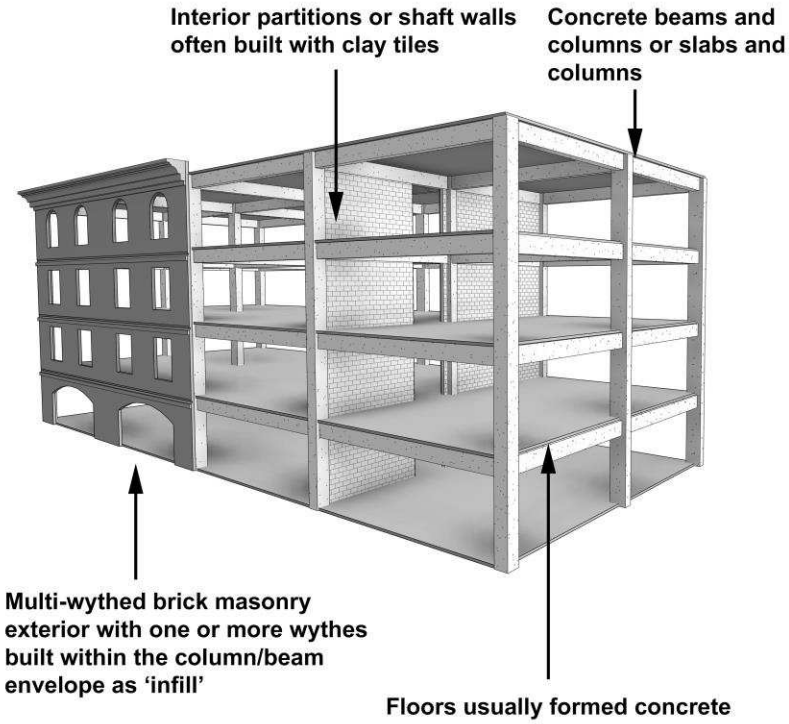


Figure B.10: Concrete frame with infill masonry shear walls (CIW) building (reproduced from FEMA 547, FEMA, 2006)

B.3.4 Precast concrete walls (PCW)

Most of these use the tilt-up construction method. In traditional tilt-up *buildings*, concrete wall panels are cast on the ground and then tilted upward into their final position (Figure B.11). More recently, wall panels have begun to be fabricated off-site and trucked in. The wall panels are welded together or held in place by cast-in-place columns or steel columns, depending on the region. The floor and roof beams are often glue-laminated wood or steel open-webbed joists attached to the tilt-up wall panels; these panels may be loadbearing or non-loadbearing, depending on the region. These *buildings* tend to be low-rise industrial or office *buildings*. In 2015, the NBC officially included tilt-up construction for the first time as a new type of concrete SFRS.

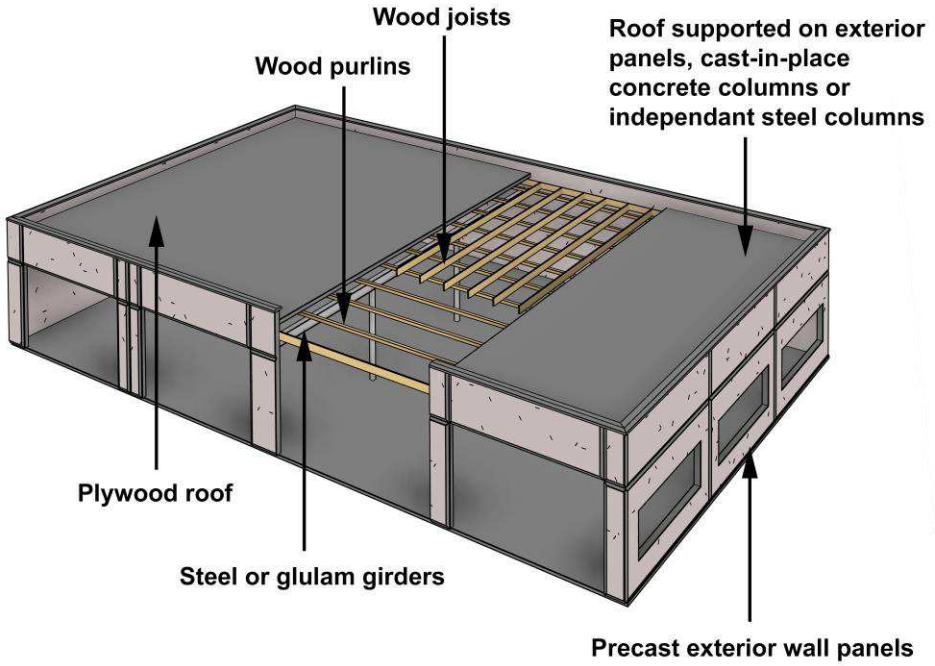


Figure B.11: Precast concrete walls (PCW) building (reproduced from FEMA 547, FEMA, 2006)

Many tilt-up *buildings* do not have sufficiently strong connections or anchors between the walls and the roof and floor diaphragms. During an earthquake, weak anchors are pulled out from the walls, causing the floors or roofs to collapse. The connections between concrete panels are also vulnerable to failure. Without these connections, the building loses much of its SFRS capacity. For these reasons, many tilt-up *buildings* were damaged in the 1971 San Fernando earthquake. Since 1975, tilt-up construction practices have changed in Canada's seismic regions, requiring positive wall-diaphragm connection. However, a large number of these older, pre-1970s vintage tilt-up *buildings* still exist, and have not been retrofitted to correct this wall anchor defect. Damage to these *buildings* was again observed in the 1987 Whittier, California earthquake. These *buildings* are a prime source of seismic hazards. In areas of low or moderate seismicity, inadequate wall anchor details continue to be employed. Severe ground shaking in such an area may produce major damage in many tilt-up *buildings*.

B.3.5 Precast concrete frame (PCF)

Precast concrete frame construction, first developed in the 1930s, was not widely used until the 1960s. The precast frame (Figure B.12) is essentially a post-and-beam system in concrete where columns, beams, and slabs are prefabricated and assembled on site. Various types of members are used. Vertical load carrying elements may be Ts, cross shapes or arches, and are often more than one storey in height. Beams are often Ts and double Ts, or rectangular sections. Pre-stressing of the members, including pre-tensioning and post-tensioning, is often used.

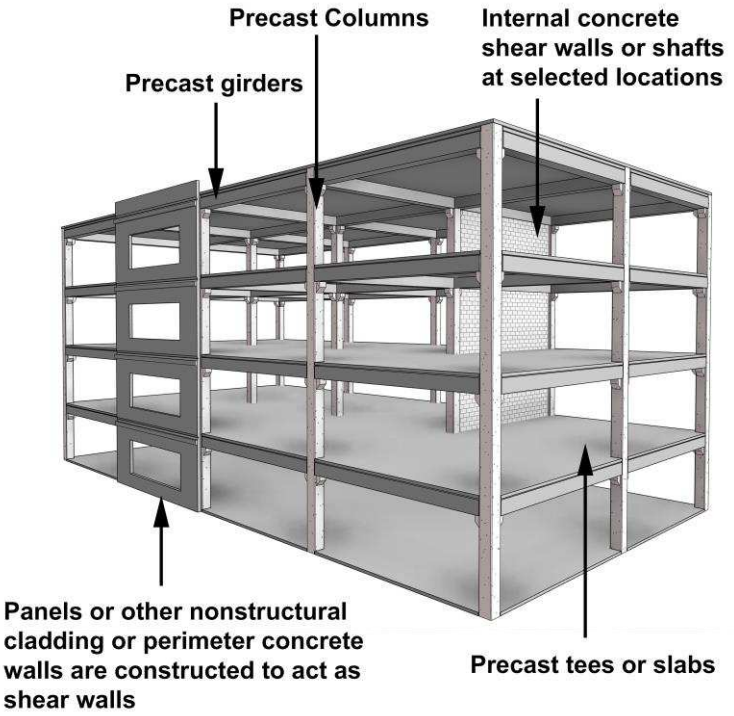


Figure B.12: Precast concrete frame (PCF) building (reproduced from FEMA 547, FEMA, 2006)

The earthquake performance of this structural type varies greatly and is sometimes poor. This type of building can perform well if the details used to connect the structural elements have sufficient strength and ductility. Some of the problem areas specific to precast frames are as follows:

1. Poorly designed connections between prefabricated elements may fail;
2. Accumulated stresses or gaps may result because of shrinkage and creep;
3. Loss of vertical support may occur due to inadequate bearing area or insufficient connection between floor diaphragm elements, beams, wall panels, and columns;
4. Corrosion of metal connectors between prefabricated elements may occur; and
5. Connections at bases of precast columns and wall panels may be inadequate.

Structures of this type that employ cast-in-place concrete shear walls for lateral load resistance should be treated as concrete shear walls (CSW).

B.4 Masonry buildings

B.4.1 **Reinforced masonry bearing walls with light wood or metal deck diaphragms (RML)**

Reinforced-masonry *buildings* (Figure B.13) are mostly low-rise perimeter bearing-wall structures, often with wood diaphragms, although steel decking is sometimes used. Floor and roof assemblies usually consist of timber joists and beams, glue-laminated beams or light steel joists.

The bearing walls consist of grouted and reinforced hollow or solid masonry units. Interior supports, if any, are often wood or steel columns, wood stud frames or masonry walls. The occupancy of this building type varies greatly from small commercial *buildings* to residential and industrial *buildings*. Generally, they are less than five storeys in height, although many mid-rise masonry *buildings* exist.

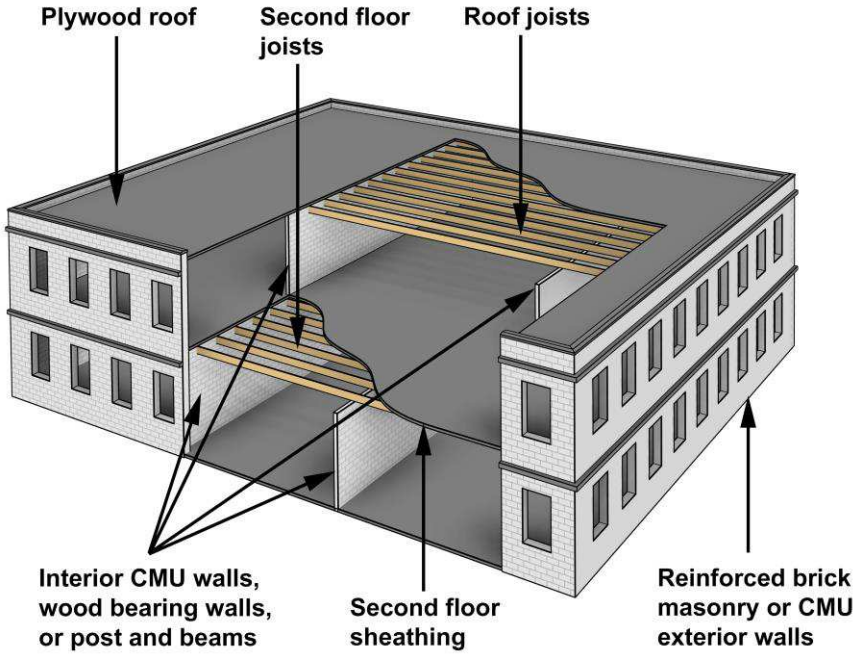


Figure B.13: Reinforced masonry bearing walls with light wood or metal deck diaphragms (RML) (reproduced from FEMA 547, FEMA, 2006)

Reinforced-masonry *buildings* can perform well in moderate earthquakes, if they are adequately reinforced and grouted, and if sufficient diaphragm anchorage exists. A major problem is control of workmanship during construction. Poor construction practices may result in ungrouted and unreinforced walls. These conditions led to several collapses in the 1964 Alaska earthquake. Where construction practices are adequate, insufficient reinforcement may be the cause of heavy wall damage. A lack of positive connection of the floor or roof diaphragm to the wall is an issue.

B.4.2 Reinforced masonry bearing walls with concrete diaphragms (RMC)

Reinforced-masonry *buildings* also have rigid concrete diaphragms (floor and roof construction). A typical building model of this type is illustrated in Figure B.14. The rest of the information is similar to that presented for RML. The poor anchorage and connections of precast concrete diaphragms may be responsible for earthquake-related damage.

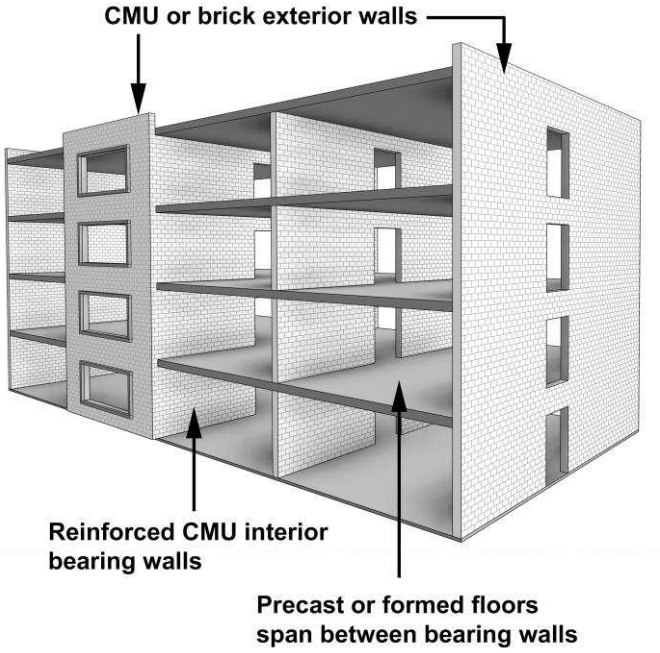


Figure B.14: Reinforced masonry bearing walls with concrete diaphragms (RMC) building (reproduced from FEMA 547, FEMA, 2006)

B.4.3 Unreinforced masonry bearing wall building (URM)

Most unreinforced masonry (URM) bearing-wall structures in Western Canada and Quebec were built before the 1940s; however, this construction type was built in some jurisdictions having moderate or high seismicity until the late 1940s or late 1950s. These *buildings* (Figure B.15) usually range from one to six storeys in height and typically function as commercial, residential, and industrial *buildings*. The construction varies according to the type of use, although wood floor and roof diaphragms are common in older *buildings*. Smaller commercial and residential *buildings* usually have light wood floor or roof joists supported on the typical perimeter URM wall and interior load-bearing wood partitions. Larger *buildings*, such as industrial warehouses, have heavier floors and interior columns, usually of wood. The bearing walls in these industrial *buildings* tend to be thick, often as much as 610 mm or more at the base. Wall thicknesses in residential *buildings* range from 230 mm on upper floors to 460 mm on lower floors.

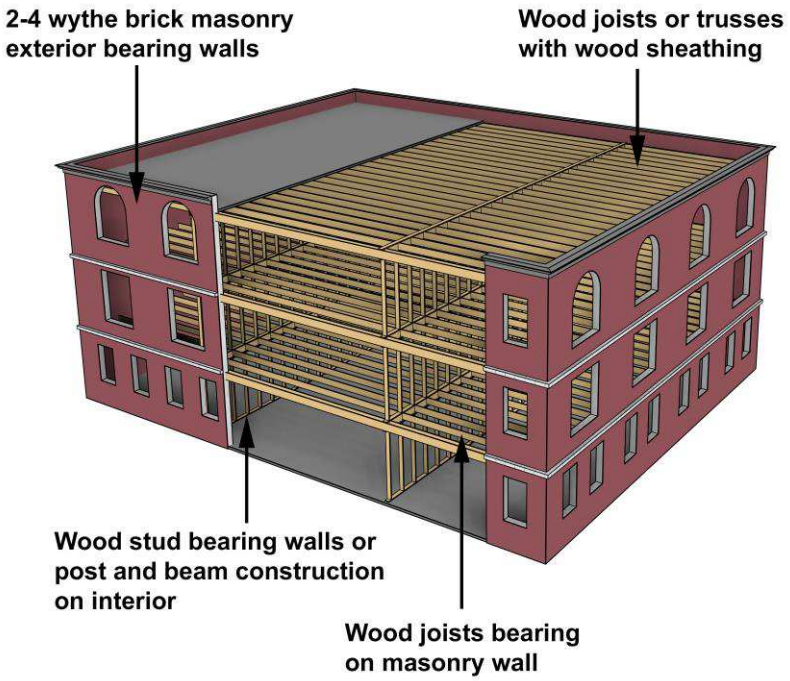


Figure B.15: Unreinforced masonry bearing wall building (URM) building (reproduced from FEMA 547, FEMA, 2006)

Unreinforced masonry structures are recognized as perhaps the most hazardous type of structure. They have been observed to fail in many modes during past earthquakes. Typical problems are as follows:

1. Insufficient anchorage – Because walls, parapets and cornices are not positively anchored to the floors, they tend to fall out. The collapse of bearing walls may lead to major building collapses. Some of these *buildings* have anchors either as a part of the original construction or as a retrofit. These older anchors exhibit questionable performance;
2. Wall slenderness – Some of these *buildings* have tall storey heights and thin walls. This condition, especially in non-load-bearing walls, will result in buckling out-of-plane under severe lateral load. Failure of a non-load bearing wall represents a falling hazard, whereas the collapse of a load-bearing wall will lead to partial or total collapse of the structure;
3. Low shear resistance – The mortar used in these older *buildings* is often made of lime and sand, with little or no cement, and has very little shear strength. The bearing walls will be heavily damaged and may collapse under large loads; and
4. Excessive diaphragm deflection – Because most of the floor diaphragms are constructed of wood sheathing, they are very flexible and permit large out-of-plane deflection at the wall transverse to the direction of the force. The large drift, occurring at the roof line, may cause the masonry wall to collapse under its own weight. This problem is less serious in medium seismic zones.

B.5 Other buildings

B.5.1 Cold-formed steel buildings

Cold-formed buildings have cold-formed steel light-frame walls supporting the majority of the lateral loads (Figure B.16). Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 600 mm apart, wood or cold-formed steel trusses, structural steel or cold-formed steel beams, and structural steel or cold-formed steel columns. The first floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. Seismic forces are resisted by wooden or metal deck diaphragms and wood structural, steel sheet sheathed, or diagonal strap-braced stud shear walls.

Figure B.16: Cold-formed steel (CFS) buildings (photo from BuildUsingSteel, 2018)

B.5.2 Manufactured homes

The term “manufactured homes” encompasses “modular *buildings*” (Figure B.17 (a)) and “mobile homes” (Figure B.17 (b)). Modular buildings are factory built in units or modules, but do not have permanent chassis or axles and must be transported to the site on flatbed trucks. The modules are set in place using a crane, and then are joined together. Modular buildings are typically placed on traditional permanent concrete foundations, with or without basements. Mobile homes are prefabricated, permanently attached to a chassis, and can be relocated either by being towed or transported on a trailer. Mobile homes are generally placed on concrete blocks or metal supports. Unlike mobile homes, modular buildings cannot be moved once they are built. Examples of manufactured homes include temporary or permanent dwellings and portable school *buildings*.

(a) Modular building (photo from Karmod, 2017).

(b) Mobile home (photo from Morgan, 2012).

Figure B.17: Manufactured homes (MH)

Numerous studies showed that mobile homes performed significantly worse than conventional wood-frame dwellings in the 1994 Northridge earthquake (SSC, 1995). The main weakness in mobile homes during seismic action is related to their shifting off their foundations. However, proper bracing can be provided to ensure a manufactured home will not fall off its supports.

From a life safety standpoint, the poor seismic performance of mobile homes' supports represents a greater danger than in wood frame dwellings. However, similarly to unreinforced masonry *buildings,* the extent of danger is still not higher than for other structures. Even when manufactured homes are thrown off their supports, they do not collapse, and, although occupants experience violent shocks, they are usually non-life threatening. It is also expected that injuries in unbraced mobile homes are higher than in conventional wood-frame dwellings, due to occupants and furniture being thrown about and the occasional penetration of steel jack stands through the floors. Serious threat to life may occur when exit doors in mobile homes become stuck in their closed positions, particularly in the event of a post-earthquake fire. Fires are usually a result of severe damage to gas lines that occurs when mobile homes are shifted several feet in an earthquake. Based on the observed mobile home damage subsequent to the 1994 Northridge earthquake, new requirements appeared in 1995 in the Mobilehome Parks Act, the California Health and Safety Code, and the California Code of Regulations. The new requirements included installing an approved ERB system and providing a flexible gas connection fitted with an automatic shutoff device.

Prefabricated structures that have permanent anchorage to the foundation are not considered to be MH *buildings*. The appropriate building type for properly anchored *buildings* should be selected based on the type of SFRS in the structure.

This page is intentionally left blank.

APPENDIX C EVOLUTION OF NORTH AMERICAN MODEL BUILDING CODES THROUGHOUT THE LAST CENTURY

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
1941	• **First** NBC edition • Contained seismic design provisions in Appendix H • Seismic provisions were based on the 1935 edition of Uniform Building Code (UBC) • Seismic provisions were not mandatory	$V = CW$ $0.02 \leq C \leq 0.05$ (depending on soil bearing capacity)
1953	• **First** qualitative seismic zoning map • Seismic provisions were moved to the main body of the code • Recognition of building flexibility influence on seismic force levels (first introduced by 1943 Los Angeles Building Code) • Introduced four seismic zones based on relative seismicity intensities	$F = CW$ Zone 0 : $C = 0$ Zone 1: $C = 0.15/(N+4.5)$ Zone 2: $C = 0.30/(N+4.5)$ Zone 3: $C = 0.60/(N+4.5)$ N = No. storeys above the storey under consideration
1960	• Seismic design provisions were similar to 1953 • **First** Canadian code to address the need for consideration of torsional effects	
1965	• Seismic zoning maps similar to the NBC 1953 • **First** introduction of: importance factor, foundation factor, torsional effects (based on the 1966 Mexican building code) • **First** Linear distribution of seismic force proportional to floor height and weight (similar to SEAOC Blue Book1959), or **first** recognition of triangular distribution of design acceleration over building height based on the SEAOC Blue Book 1959 • **First** torsional design provisions (based on the 1966 Mexican building code) • **First** recognition of structural ductility (by using *C* factor) • **First** recognition of dynamic analysis as an alternative to static analysis • Working stress deign, except for concrete structures, where alternatively ultimate strength design was permitted (based on ACI 1963). • Ultimate load combination including earthquake was: - U = 1.35(D + L + E)	$V = RCIFSW$
1970	• **First** probabilistic seismic zoning map • 100-year return period • Structural flexibility factor linked to fundamental period • Fundamental period equations similar to those in SEAOC Blue Book 1959 • Similar to SEAOC Blue Book 1959, higher mode effects accounted through: application of portion of lateral force at roof, reduction of overturning moment • **First** Canadian code where structural response is a function of fundamental period	$V = 0.25R(KCIFW)$

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
	• Seismic design and detailing provisions for ductile moment resisting frames and ductile flexural walls NOT provided until 1973 when CSA A23.3-73 was introduced • Seismic design and detailing provisions for "ductile moment resisting space frame" in CSA A23.3-73 and its supplement No. 1 in 1977 are essentially the same as those in Appendix A of ACI 1971, which remained unchanged in ACI 1977 • Seismic design and detailing provisions for "ductile flexural walls" in CSA A23.3-73 are more comprehensive than those in ACI for "special shear walls" • The definition of "flexural wall" in the A23.3-73 was changed in A23.3-77 • The maximum allowable nominal shear stress in "flexural walls" specified in CSA A23.3-73 was reduced in CSA A23.3-77 • CSA A23.3-73 requires "ductile flexural walls" "to have adequate ductility and energy absorption capacity in accordance with generally accepted principles." CSA Commentary in 1974 suggests a simple interim procedure for satisfying this requirement (having minimum sectional ductility of 3) • *R*, *I*, *F* and W remained unchanged from the NBC 1965 • Alternative ultimate strength design was permitted, with ultimate load combinations including earthquake as: - U = 1.15D + 1.35(L + E) - U = 1.5D + 1.8E - U = 0.9D + 1.35E • Horizontal force factor "C_p" for parts or portions of structures introduced	
1975	• Seismic zoning maps similar to the NBC 1970 • *AS* in the base shear equation in NBC 1975 was calibrated to be 20% less than 0.25*RC* in the base shear equation in NBC 1970 partly to counteract the effect of the increase in overturning moment reduction factor. Thus, seismic design force is related to traditional levels of seismic design force, rather than that produced by 100-year return period earthquake. • The effect of higher modes on overturning moment was adjusted to be less (less reduction factor – closer to 1.0 – for overturning moment) • For irregular structures, Commentary K recommended dynamic analysis • Commentary K in the NBC 1975 introduced structural ductility factor (μ) for dynamic analysis scaling purposes • Construction type factor (*K*) reflected the influence of construction type on damping, ductility and/or energy absorption capacity • Commentary K in the NBC 1975 recommended use of SRSS method for combining design forces in dynamic analysis from each mode (modal analysis) • *I*, *F* and *W* remained unchanged from the NBC 1965 • Low seismic base shear values based on dynamic analysis due to: - No upper bound to fundamental periods based on computer models ("bare structures" without considering non-structural components)	$V = ASKIFW$

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
	- Since μ does not match 1/K, the dynamic analysis scaling based on factor of μ • Alternative ultimate strength design was permitted similar to CSA A23.3-73, as: - U = 0.75(1.4D + 1.7L + 1.8E) - U = 1.4D + 1.8E - U = 0.9D + 1.4E • Alternative limit state design was permitted in the NBC 1975, with ultimate load combinations as: - U = 1.25D + 0.7(1.5L + 1.5E) - U = 1.25D + 1.5E - U = 0.85D + 1.5E Limit state design was used in CSA S16.1-74	
1977	• Same seismic design provisions and seismic zoning maps as the NBC 1975 • Introduced minimum 90% of static analysis procedure base shear for dynamic analysis base shear (in order to not stray far from the traditional levels of earthquake resistance) • Static load procedure same as that in the NBC 1975	
1980	• Introduced SI units • Same seismic zoning map as the NBC 1970 • Seismic response factor (S) was changed to: $S = 0.5/T^{0.5} \leq 1.0$ • Limit state load factors and combination factors remained the same as those in the NBC 1977 • For concrete structures, ultimate strength procedure from the NBC 1977 design was permitted in CSA A23.3-77.	
1985	• <u>New seismic zoning maps</u> (based on point source model) introduced based on ATC-3 document • <u>New</u> return period of 475 years was introduced • Seismic zones increased from four to seven zones • Recognized that the spectral shape not similar to California, and was different in Western and Eastern Canada • vS in the base shear equation is equivalent to the spectral acceleration • $S = 0.44$ for $Z_a/Z_v = 1$ chosen to calibrate the base shear values to those in the NBC 1980 • **<u>First time</u>** permitted use of period based on modal analysis limited to 1.2 times empirical value of $T = 0.09h_n/D_s^{0.5}$ • Introduced minimum 100% of static analysis procedure base shear for dynamic analysis base shear • Accidental torsional eccentricity increased to $0.10D$	$V = \nu SKIFW$

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
	• Dynamic analysis was required if centre of mass and rigidity do not line up approximately (not permitting to double the torsional moments as an alternative) • Limit state design was used in CSA A23.3-84 • Limit state design permitted for all materials: - U = 1.25D + 0.7(1.5L + 1.5E) - U = 1.25D + 1.5E - U = 0.85D + 1.5E	
1990	• Similar seismic zoning maps as in the NBC 1985 • Replacement of *K* factor by *R* factor (force modification factor) • Use of load factor of 1.0 for seismic forces • Calibration factor (*U*) introduced to "maintain design base shears at same level of protection for *buildings* with good to excellent capability of resisting seismic loads consistent with *R* factors used) – similar to the NBC 1985 level (varying between 1.0 for unreinforced masonry to 4.0 for ductile moment resisting space frames) • NBC 1990 requires seismic detailing and design be in accordance with CSA standards for steel, concrete, timber, and masonry, consistent with the chosen *R* factor • Limit state design permitted as: - U = 1.25D + 0.7(1.5L + 1.0E) - U = 1.25D + 1.0E - U = 0.85D + 1.0E • Inter-storey drift limitations of $0.01h_n$ for post-disaster *buildings*, and $0.02h_n$ for all other *buildings* were introduced	$V = \frac{U(vSIFW)}{R}$
1995	• Additional *R* factors: nominally ductile steel plate shear walls; ordinary steel plate shear walls; ductile coupled walls; reinforced masonry walls with nominal ductility • New expressions for fundamental periods • New torsional eccentricity expressions • Introduced minimum 80% of the base shear by static analysis procedure for the base shear by dynamic analysis • Companion load format was adopted for load combinations as: - U = 1.0D + 1.0E - For storage occupancies U = 1.0D + 1.0E + 1.0L - For other occupancies U = 1.0D + 1.0E + 0.5L	$V = \frac{U(vSIFW)}{R}$
2005	• **First** Uniform Hazard Spectrum (UHS) approach for giving site-specific response spectral acceleration for numerous locations in Canada rather than seismic zoning map	$V = \frac{S(T_a)M_v I_E W}{R_d R_o}$ $V \geq \frac{S(2.0)M_v I_E W}{R_d R_o}$

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
	• **First time** 2475-year return period • Included the requirements for complete load paths • Introduced definitions of irregular structures and special design requirements associated with these irregularities • Adopted a new site classification system to define six Site Classes designated as A through F Mandatory dynamic analysis for structures with certain irregularities • Introduction of two separate force modification factors: - Ductility-related force modification factor (R_d) reflecting structural capability for energy dissipation through inelastic behavior - Overstrength-related force modification factor (R_o) accounting for the dependable portion of reserved strength in a structure • Upper limits to analytical fundamental periods based on dynamic analysis to ensure general agreement with typical measured periods on existing structures • Simplified determination of torsional effects by eliminating the factor on eccentricity e_x, enabling designer accounting for torsion directly including accidental torsion through 3-D analysis and shifting mass centre by $\pm 0.1 D_{nx}$ • Introduced torsional sensitivity factor B = $\delta_{max}/\ \delta_{avg}$ • Building height restrictions depending on $I_E F_a S_a(0.2) > 0.35$ g • Introduced minimum of: - 80% of static analysis procedure base shear for dynamic analysis base shear for regular *buildings*, otherwise 100% - 100% of static analysis procedure base shear for dynamic analysis base shear for irregular *buildings* • Inter-storey drift limitations of $0.01 h_n$ for post-disaster *buildings*, $0.02 h_n$ for schools and $0.025 h_n$ for all other *buildings* • Building separation requirements were revised to "the square root of the sum of the squares" of • Introduded additional foundation provisions • Load combinations was: - U = 1.0D + 1.0E - For storage occupancies U = 1.0D + 1.0E + 1.0L + 0.25S - For other occupancies - U = 1.0D + 1.0E + 0.5L + 0.25S	
2010	• Seismic hazard values for all but western localities were recalculated using an improved fit to ground motion relations used in 2005 • Change was made to the minimum lateral seismic force for walls, coupled walls, and wall-frame systems • Additional SFRS systems: Cold-formed steel structures, steel structures with buckling-restrained braced frames • Additional restrictions for post-disaster *buildings* disallowing vertical stiffness irregularities	$V_{max} \geq V \geq V_{min}$ $V = \frac{S(T_a) M_v I_E W}{R_d R_o}$ For wall systems: $V_{min} = \frac{S(4.0) M_v I_E W}{R_d R_o}$ For frame systems:

Table C.1: Evolution of the NBC from 1941 to 2015 for seismic design provisions

NBC version	Unique characteristics	Base shear equation
	• Changes to higher mode effect factor M_v • Seismic design provisions are exempted for *buildings* with $S(0.2) \leq 0.12$ g • S_a values changed to represent the mean (rather than median) value	$V_{min} = \frac{S(2.0)M_v I_E W}{R_d R_o}$ For *buildings* located on *Site Classes* other than F, and having SFRS with $R_d \geq 1.5$: $V_{max} = \frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o}$
2015	• Updated seismic hazard values across Canada • Increased foundation factors (F_a and F_v) from two (for periods 0.2 and 1.0 second) to six (for periods 0.2, 0.5, 1.0, 2.0, 5.0, and 10 second) • Introduced site coefficients for PGA and PGV • Site coefficients are functions of reference peak ground acceleration PGA_{ref} • No exemption for low seismic zones but introducing simplified seismic design procedure for *buildings* where $I_E.F_s.S_a(0.2)$ and $I_E.F_s.S_a(2.0)$ are less than 0.16 g and 0.03 g • Introduced gravity-induced lateral demand irregularity • New seismic provisions for: *buildings* with flexible diaphragms, inclined columns, passive energy dissipation, glazing systems, steel pallet storage racks, base isolation, elevators • Revisions in ductility- and overstrength-related modification factors (R_d and R_o) for concrete and masonry • New ductility- and overstrength-related modification factors (R_d and R_o) for tilt-up construction in reinforced concrete • Changes in foundation design provisions • Seismic design provisions for timber construction more than four storeys tall • S_a values changed to represent the mean (rather than median) value • Introduced a new flowchart for seismic evaluation and upgrading of existing buildings in Commentary L	$V_{max} \geq V \geq V_{min}$ $V = \frac{S(T_a)M_v I_E W}{R_d R_o}$ For walls, couples walls and wall-frame systems: $V_{min} = \frac{S(4.0)M_v I_E W}{R_d R_o}$ For moment resisting frames, braced frames and other systems: $V_{min} = \frac{S(2.0)M_v I_E W}{R_d R_o}$

Table C.2: Evolution of the UBC and IBC for seismic design provisions

UBC version	Unique characteristics	Base shear equation
1927	• Contained seismic design provisions in the Appendix of the code • No seismic hazard maps were in existence	$F = 0.075\,W$ ($F = 0.1W$ on poor soils)
1935	• **First** seismic zone map of the eleven western states, developed in 1928 by N.H. Heck • Applied 50% of live load in addition to dead load	$F = CW$ $0.02 \leq C \leq 0.05$

Table C.2: Evolution of the UBC and IBC for seismic design provisions

UBC version	Unique characteristics	Base shear equation
1937	-	(depending on soil bearing capacity)
1940	-	
1943	-	
1946	-	
1949	• **First National Seismic Probability Zone Map** of the U.S. compiled in 1948 by the U.S. Coast and Geodetic Survey (USCGS) • Contained four Seismic Zones (0-3) with Zone 0 corresponding to no damage and Zone 3 indicating major damage • Recognition of building flexibility influence on seismic force levels	$F = CW$ Zone 0 : $C = 0$ Zone 1: $C = 0.15/(N+4.5)$ Zone 2: $C = 0.30/(N+4.5)$ Zone 3: $C = 0.60/(N+4.5)$ N = No. storeys above the storey under consideration
1952	-	
1955	-	
1958	• Slight change in the 1949 UBC seismic zone map	
1961	• Seismic design provisions moved to the main body of the code • Lateral force system structural factors included • Fundamental period of vibration included • Horizontal force factor "C_p" for parts or portions of structures introduced • **First** code to address the need for consideration of torsional effects and lateral deflections	$V = ZKCW$
1964	• **First** code to introduce some ductile detailing requirements	
1967	-	
1970	• **First** recognition of dynamic analysis as an alternative to static analysis	
1973	• Introduced the impact of irregular parameters • **First** required use of the special frame details in regions of highest seismicity	
1976	• Addition of Seismic Zone 4 • Site-structure resonance effect included • Building importance factor included • **First** "modern" building code for concrete moment frame construction due to its improved seismic requirements	$V = ZIKCSW$
1979	-	
1982	-	
1985	• Three soil profiles introduced	

Table C.2: Evolution of the UBC and IBC for seismic design provisions

UBC version	Unique characteristics	Base shear equation
1988	• Seismic zones modified: division of Seismic Zone 2 into 2A and 2B • Use of a *Z* factor roughly indicative of the peak acceleration on rock, corresponding to 475-year return period • Soil profiles expanded • Building irregularities included • Hight limits for various structural systems in Seismic Zones 3 amd 4 • Numerical coefficient R_w for various structural systems introduced	$V = \frac{ZICW}{R_w}$
1991	-	
1994	-	
1997	• A new site classification system adopted to define Site Classes designated as A through F • System redundancy factor introduced • Additional SFRSs introduced • Vertical component of ground shaking included • Seismic provisions based on strength-level design	$V = \frac{C_v IW}{RT}$
IBC 2000	• 1997 UBC replaced by IBC 2000 • Spectral response accelerations introduced to determine seismic hazards • Safety concept redefined • Seismic design categories (SDC) introduced • System response modification factors expanded • **First time** 2475-year return period for maximum considered earthquake	$V = C_s W$
2003	-	
2006	• Referenced ASCE/SEI 7-05 • Additional SFRSs introduced	
2009	-	
2012	• Risk categories introduced • Risk-targeted maximum considered earthquake introduced • Commentary C1 provided tables for target reliabilities (conditional probabilities of failure) for structural stability and ordinary noncritical structural members caused by earthquake for various risk categories	
2015	-	
2018	• Tables including target reliabilities (conditional probabilities of failure) for structural stability and ordinary noncritical structural members caused by earthquake for various risk categories are moved to the main body of the standard • Site coefficients revised	