Level 2: Semi-Quantitative Seismic Risk Screening Tool (SQST) for existing buildings. Part 2: supporting technical documentation

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Level 2 – Semi-Quantitative Seismic Risk Screening Tool (SQST) for Existing Buildings

Part 2: Supporting Technical Documentation

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

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LEVEL 2 – SEMI-QUANTITATIVE SEISMIC RISK SCREENING TOOL (SQST) FOR EXISTING BUILDINGS

PART 2: SUPPORTING TECHNICAL DOCUMENTATION

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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. Volume II 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 2 – SQST*, Part 2: Supporting Technical Documentation, complements Part 1: User's Guide. It provides details and assumptions for developing the seismic screening methodology in the *Level 2 – SQST*. Specifically, it describes the determination of basic scores, score modifiers, and score thresholds for structural and non-structural components. Furthermore, it provides additional information to facilitate understanding of *seismic risk* in *existing buildings* and structural response to earthquakes of different *model building types*.

A key goal is to provide sufficient information that a professional with some basic knowledge of probability and structural engineering would use to calculate the basic scores, score modifiers, and score thresholds provided in *Level 2 – SQST* screening forms in Part 1: User's Guide.

1.3 Organization of the supporting technical documentation

Chapters 2 and 3 present a detailed description of the *Level 2 – SQST* methodology for *seismic risk* screening of structural and non-structural components in *existing buildings*. Specifically, the structural scoring system as part of the *Level 2 – SQST* methodology quantitatively evaluates the structural *seismic risk* by calculating the probability of collapse, and the non-structural scoring system, as part of the *Level 2 – SQST* methodology, qualitatively evaluates the global *seismic risk* caused by non-structural components by calculating the seismic demand of the most critical non-structural components. Structural and non-structural component scores are determined based on the sum of a basic score and a series of score modifiers, and are compared separately against acceptable structural and non-structural thresholds to determine if the *building's seismic risk* exceeds the acceptable *seismic risk.*

Chapter 4 provides details regarding the implementation of the *Level 2 – SQST* to seventeen (17) *existing buildings* provided by PSPC.

2.0 METHODOLOGY FOR SEISMIC RISK SCREENING OF EXISTING BUILDINGS: STRUCTURAL SCORING SYSTEM

2.1 General

The *Level 2 – Semi-Quantitative Seismic Risk Screening Tool (SQST)* is a detailed *seismic risk* screening tool that aims to quickly and inexpensively assess the *seismic risk* of *existing buildings*. The *Level 2 – SQST* methodology consists of a structural scoring system and a non-structural scoring system, based on which the structural and non-structural component scores of *existing buildings* are calculated separately. The scores are then compared separately against the acceptable thresholds to determine whether a *building*'s *seismic risk* exceeds the acceptable *seismic risk*. *Buildings* with potential unacceptable *seismic risk* are prioritized for *Level 3 – SEG* using a ranking procedure (see Part 1: User's Guide).

The structural scoring system presented in this section is largely based on FEMA P-154 methodology (FEMA 2015) (see APPENDIX C); however, it incorporates a number of new features, namely:

(1) Additional *building* attributes, including *original building importance*, *building deterioration* and age, and *remaining occupancy time*;

(2) New *seismic zones* reflecting Canadian *seismicity* developed from empirical relationships between spectral response accelerations at short and long periods for design ground motions and the Modified Mercalli Intensity – MMI scale;

(3) Five levels of *consequences of failure* defined to describe the *consequences of failure* of *buildings* associated with the risk to human lives; and

(4) Structural thresholds determined for different levels of *consequences of failure*.

The structural scoring methodology determines the structural scores of *existing buildings* based on probability of collapse. The probabilities of collapse of *existing buildings* are calculated by incorporating generic *building* capacity curves, fragility curves, and collapse factors. *Building* capacity curves and fragility curves are developed for sixteen *model building types*. These *model building types* are used to group *existing buildings* into different categories. In addition, a total of six *seismic zones* are defined to group all locations across Canada into different categories. *Seismic zone* is a key parameter in developing *building* capacity and fragility curves. The structural scoring procedure begins by identifying the *model building type* and determining the applicable *seismic zone*. A structural basic score is determined by calculating the probability of collapse for a given *model building type* and code level earthquake (CLE) corresponding to a specified *seismic zone*. Subsequently, one or more applicable score modifiers are calculated for applicable *building* conditions. The structural score is determined by adding the structural basic score to the applicable score modifiers, and is then compared with a specified structural threshold to determine whether

Level 3 – SEG should be initiated. *Buildings* with structural scores that are lower than their applicable structural thresholds are flagged for *Level 3 – SEG*. The structural thresholds are determined based on acceptable probabilities of failure prescribed in applicable *building* codes and standards.

Details for the development of the structural scoring methodology, including the suitability of FEMA P-154 methodology for Canada, description of the structural scoring methodology, determination of structural basic scores and score modifiers, and determination of acceptable structural thresholds are provided below.

2.2 Suitability of the FEMA P-154 methodology for Canada

Based on the literature review of existing seismic screening methodologies (see APPENDIX A), the *Level 2 – SQST* project team evaluated the suitability of the FEMA P-154 methodology for *building* seismic design and construction practices in Canada. Then, the FEMA P-154 methodology was customized with additional features, to develop the structural scoring methodology. Customization was mainly focused on the following key aspects: (1) *model building type*, (2) key *building* code edition, (3) seismic design factors and coefficients, and (4) *seismic zones*. These aspects are essential for determining *building* capacity and fragility curves and thus are crucial for determining structural basic scores and score modifiers.

2.2.1 Model building type

A total of sixteen *model building types* (MBTs) are considered in the *Level 2 – SQST* (see Table 2.1). The MBTs are adopted from the *Level 1 – Preliminary Seismic Risk Screening Tool* (*PST*), but exclude the cold-formed steel (CFS) MBT. The main reason for excluding the CFS is there is no available data to establish *building* capacity and fragility curves for this MBT. Compared to the 1993 NRC screening manual (NRC 1993a), one new MBT – manufactured homes (MH) – is included. The main reasons to include MH buildings are: (1) FEMA P-154 provided parameter values for assessing the probabilities of collapse of MH *buildings*, and (2) the Canadian Standards Association (CSA) Standard for Manufactured (Mobile) Home Construction (CSA Z240.10.1) has been explicitly referenced by the NBC since its 2005 edition (i.e. the NBC 2005). A mapping is presented in Table 2.1 to illustrate the conversion between the two *building* classification systems in the U.S. and Canada. Justification for the mapping can be found in Part 2: Supporting Technical Documentation of *Level 1 – PST*.

Table 2.1: Model building types in *Level 2 – SQST*

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

It is noted that FEMA P-154 defines W2 *building* type primarily in terms of occupancy. W2 includes both light frame construction and post-and-beam construction. The NRC, on the other hand, defines wood *building* types primarily in terms of the type of SFRS. Given this, two wood *building* types, i.e., wood light frame (WLF) and wood post-and-beam (WPB), are considered in the *Level 2 – SQST*.

The NBC 2015 defines different types of SFRS in terms of main construction materials. A comparison of the MBTs in *Level 2 – SQST* and the SFRSs in the NBC 2015 is presented in Table 2.2. The *Level 2 – SQST* project team decided to use sixteen MBTs rather than adopting the SFRSs because: (1) the MBTs are consistent with those used in the 1993 NRC screening manual; (2) *building* capacity and fragility curves were developed for the MBTs rather than the SFRSs; and (3) it is difficult to determine the level of ductility of a SFRS in a quick visual screening procedure used in *Level 2 – SQST*.

Table 2.2: SFRS as per Table 4.1.8.9 of Part 4 of the NBC 2015 and equivalent model building types in *Level 2 – SQST*

Type of SFRS	R_d	R_o	Restriction: Cases where $I_E F_a S_a(0.2)$ < 0.2	≥ 0.2 to ≤ 0.35	≥ 0.35 to ≤ 0.75	> 0.75	Cases where $I_E F_v S_a(1.0)$ > 0.3	Model Building Types
Steel Structures Designed and Detailed According to CSA S16								
Ductile moment-resisting frames	5	1.5	NL	NL	NL	NL	NL	**SMF**
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL	
Limited ductility moment-resisting frames	2	1.3	NL	NL	60	30	30	
Moderately ductile concentrically braced frames								**SBF**
Tension-compression braces	3	1.3	NL	NL	40	40	40	
Tension only braces	3	1.3	NL	NL	20	20	20	
Limited ductility concentrically braced frames								
Tension-compression braces	2	1.3	NL	NL	60	60	60	
Tension only braces	2	1.3	NL	NL	40	40	40	
Ductile buckling-restrained braced frames	4	1.2	NL	NL	40	40	40	
Ductile eccentrically braced frames	4	1.5	NL	NL	NL	NL	NL	
Ductile plate walls	5	1.6	NL	NL	NL	NL	NL	**N/A**
Limited ductility plate walls	2	1.5	NL	NL	60	60	60	
Conventional construction of moment-resisting frames braced frames or plate walls								
Assembly occupancies	1.5	1.3	NL	NL	15	15	15	**SMF/ SBF**
Other occupancies	1.5	1.3	NL	NL	60	40	40	
Other steel SFRS(s) not defined above	1	1	15	15	NP	NP	NP	**SIW/ SCW**
Concrete Structures Designed and Detailed According to CSA A23.3								
Ductile moment-resisting frames	4	1.7	NL	NL	NL	NL	NL	**CMF**
Moderately ductile moment-resisting frames	2.5	1.4	NL	NL	60	40	40	
Ductile coupled walls	4	1.7	NL	NL	NL	NL	NL	**CSW**
Moderately ductile coupled walls	2.5	1.4	NL	NL	NL	60	60	
Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL	NL	
Moderately ductile partially coupled walls	2	1.4	NL	NL	NL	60	60	
Ductile shear walls	3.5	1.6	NL	NL	NL	NL	NL	
Moderately ductile shear walls	2	1.4	NL	NL	NL	60	60	
Conventional construction								
Moment-resisting frames	1.5	1.3	NL	NL	20	15	10	**CMF**
Shear walls	1.5	1.3	NL	NL	40	30	30	**CSW**
Two-way slabs without beams	1.3	1.3	20	15	NP	NP	NP	**CMF**
Tilt-up construction								**PCF/ PCW**
Moderately ductile walls and frames	2	1.3	30	25	25	25	25	

Limited ductility walls and frames	1.5	1.3	30	25	20	20	20	
Conventional walls and frames	1.3	1.3	25	20	NP	NP	NP	
Other concrete SFRS(s) not listed above	1	1	15	15	NP	NP	NP	**SIW/ CIW/ SCW**
Timber Structures Designed and Detailed According to CSA O86								
Shear walls								
Nailed shear walls: wood-based panel	3	1.7	NL	NL	30	20	20	**WLF**
Wood-based and gypsum panels in combination	2	1.7	NL	NL	20	20	20	
Braced or moment-resisting frames with ductile connections								
Moderately ductile	2	1.5	NL	NL	20	20	20	**WPB**
Limited ductility	1.5	1.5	NL	NL	15	15	15	
Other wood- or gypsum-based SFRS(s) not listed above	1	1	15	15	NP	NP	NP	**N/A**
Masonry Structures Designed and Detailed According to CSA S304								
Ductile shear walls	3	1.5	NL	NL	60	40	40	
Moderately ductile shear walls	2	1.5	NL	NL	60	40	40	
Conventional construction								**RML/ RMC**
Shear walls	1.5	1.5	NL	60	30	15	15	
Moment-resisting frames	1.5	1.5	NL	30	NP	NP	NP	
Unreinforced masonry	1	1	30	15	NP	NP	NP	**URM**
Other masonry SFRS(s) not listed above	1	1	15	NP	NP	NP	NP	**N/A**
Cold-Formed Steel Structures Designed and Detailed According to CSA S136								
Screw-connected shear walls – wood-based panels	2.5	1.7	20	20	20	20	20	
Screw-connected shear walls – wood-based and gypsum panels in combination	1.5	1.7	20	20	20	20	20	
Diagonal strap, concentrically braced walls								**CFS**
Limited ductility	1.9	1.3	20	20	20	20	20	
Conventional construction	1.2	1.3	15	15	NP	NP	NP	
Other cold form SFRS(s) not defined above	1	1	15	15	NP	NP	NP	**N/A**

2.2.2 Key building code edition

Building design code edition is a key criterion in the *seismic risk* screening of *existing buildings*. Over the evolution of model *building* codes, more and more comprehensive and stringent seismic provisions have been adopted and enforced to reflect the advances in earthquake engineering and to address the *building* deficiencies observed in major earthquakes around the world. In this section, key *building* code editions in the U.S. are discussed first and then mapped to applicable Canadian *building* code editions.

In the U.S., the Uniform Building Code (UBC) is the model *building* code. The first edition of the UBC (i.e. 1928 UBC) was published in 1928 by the International Conference of Building Officials (ICBO) and has been superseded by the International Building Code (IBC) since the first edition of the IBC in 2000 (i.e. IBC 2000). Although the evolution of the UBC has been successive over its editions, there are two key *building* code editions: the *pre-code edition* in which seismic provisions were first adopted and enforced by the jurisdiction, and the *benchmark NBC edition* in which substantial improvements in seismic design requirements (e.g., ductile capacity for inelastic energy dissipation, seismic detailing requirements, etc.) were adopted and enforced, and could differ for each MBT (FEMA P-154 2015). Based on the *pre-code edition* and *benchmark NBC edition*, the *existing buildings* are grouped into three categories: (1) *pre-code buildings*, (2) *pre-benchmark buildings*, and (3) *post-benchmark buildings*. In FEMA P-154, basic scores ("base case") are calculated based on *pre-benchmark buildings* in moderate to high seismic zones without any deficiencies. A pre-code modifier or post-benchmark modifier is applied to a *building* designed prior to and after the *pre-code edition* or the *benchmark NBC edition*, respectively.

In Canada, the National *Building* Code of Canada (NBC) is the model *building* code. The seismic provisions were first included in Appendix H of the first edition of the NBC (i.e. the NBC 1941). It should be noted that the seismic provisions in this edition are not mandatory (NRC 1941). In the 1953 edition of the NBC (NBC 1953), the seismic provisions were moved to the main body of the code and became mandatory. Therefore, the NBC 1953 is chosen as the *pre-code NBC edition* for all model *building* types. The one exception is precast concrete wall (PCW) *buildings*, for which the NBC 1975 is chosen as the *pre-code NBC edition*. In this edition, load-bearing precast concrete elements were first required to have connections designed for earthquake loads, which are essential for ensuring structural integrity and lateral seismic resistance when subjected to earthquakes. Selection of the NBC 1975 is consistent with that in the index assignment procedure developed by Karbassi and Nollet (2008).

The *benchmark NBC editions* should capture major improvements to seismic provisions over the evolution of the NBC and referenced Canadian Standards Association (CSA) standards. Table 2.3 presents the *benchmark NBC editions* for different *model building types* for the *Level 2 – SQST* and the *Level 1 – PST*. Details regarding identification of *benchmark NBC editions* are provided in Part 2: Supporting Technical Documentation of the *Level 1 – PST* (Fathi-Fazl et al. 2019). It should be noted that *building damage* in SIW, CIW and URM *buildings* are a function of lateral seismic force instead of lateral displacement. Seismic design provisions prohibited their use in moderately high through very high *seismicity* areas. In addition, height restrictions are applied to limit their lateral displacements subjected to earthquake ground motions. Given this, the seismic resistance of these three *model building type*s designed and constructed in very low to moderate *seismicity* areas in accordance with the *benchmark NBC editions* are deemed to be adequate. As a result, *Level 1 – PST* specifies *benchmark NBC editions* for SIW, CIW, and URM *buildings* (Table 2.3). The *Level 2 – SQST*, however, does not specify *benchmark NBC editions* for SIW, CIW, and URM *buildings* since they are not expected to demonstrate ductility capacity for energy dissipation, in accordance with the NBC and referenced CSA standards. Ductility capacity is required in determining the post-benchmark modifiers in the *Level 2 – SQST*.

Table 2.3: *Benchmark NBC editions* of different model *building* types in *Level 1 – PST* and *Level 2 – SQST*

MBT	Benchmark NBC edition (*Level 2 – SQST*)	Benchmark NBC edition (*Level 1 – PST*)
WLF	2005 (≤ 4 *storeys*); 2015 (4 < *storeys* ≤ 6)	2005 (≤ 4 *storeys*); 2015 (4 < *storeys* ≤ 6)
WPB	2005	2005
SMF	2005	2005
SBF	2010 (buckling-restrained braced frames); 2005 (other)	2010 (buckling-restrained braced frames); 2005 (other)
SLF	2005	2005
SCW	2005	2005
SIW	NA	2005
CMF	2015 (two-way slabs without beams); 2005 (other)	2015 (two-way slabs without beams); 2005 (other)
CSW	2005	2005
CIW	NA	2005
PCW	2015	2015
PCF	2005	2005
RML	2005	2005
RMC	2005	2005
URM	NA	2005
MH	2005 (< 4.3 m wide and 1 *storey*) 2010 (≥ 4.3 m wide or 2-3 *storeys*)	2005 (< 4.3 m wide and 1 *storey*) 2010 (≥ 4.3 m wide or 2-3 *storeys*)

It should be emphasized that *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, 2017). This is consistent with the requirements for non-structural components in other seismic evaluation standards such as National Institute of Standards and Technology (NIST) Standards of Seismic Safety for Existing Federally Owned and Leased Buildings (NIST 2011) and the ASCE/SEI 41 Standard for Seismic Evaluation and Retrofit of Existing Buildings (ASCE 2017).

2.2.3 Seismic design factors and coefficients

Seismic design factors and coefficients are key parameters for calculating structural seismic demand and lateral deflections, which are crucial for developing *building* capacity and fragility curves. As mentioned before, many aspects of seismic design provisions in U.S. *building* codes and the NBC are essentially consistent. For brevity, the reader is directed to APPENDIX A in Part 2: Supporting Technical Documentation of the *Level 1 – PST* for the detailed evolution of NBC editions and UBC editions (superseded by the IBC since 2000).

Table 2.4 provides a comparison of seismic design formulas in the IBC 2018 and NBC 2015, which are functions of seismic design factors and coefficients, to support the adoption of *building* parameter values from FEMA P-154. Since seismic design provisions in the IBC 2018 refer to ASCE/SEI 7-16 (2016), the base shear formulas in ASCE/SEI 7-16 are included in the comparison.

Table 2.4: Base shear formulas in ASCE 7-16 and the NBC 2015

Code	IBC 2018; ASCE/SEI 7-16	NBC 2015
Base shear	$V_s = C_s W, \; C_s = \frac{S(T_a)}{(R/I_E)}$	$V = \frac{S(T_a) M_v I_E W}{R_d R_o}$

The design base shear in ASCE/SEI 7-16 can be rewritten as:

$$V_s = \frac{S(T_a) I_E W}{R} \tag{2.1}$$

where R is the response modification factor used to reduce the structure's seismic demand in the seismic design, reflecting the structural ductile behaviour during earthquake events. As shown in Figure 2.1, R is expressed as the ratio of the lateral seismic force that would develop under the specified ground motion if the structure had an entirely linear-elastic response to the prescribed design forces:

$$R = \frac{V_E}{V_s} \tag{2.2}$$

R can also be rewritten as:

$$R = \frac{V_E}{V_y} \cdot \frac{V_y}{V_s} = R_d \, \Omega \tag{2.3}$$

where R_d is a ductility modification factor and Ω is an overstrength factor. Values of R_d and Ω (same as Ω_0 in ASCE/SEI 7-16) are provided in Table 12.2.1 in ASCE/SEI 7-16.

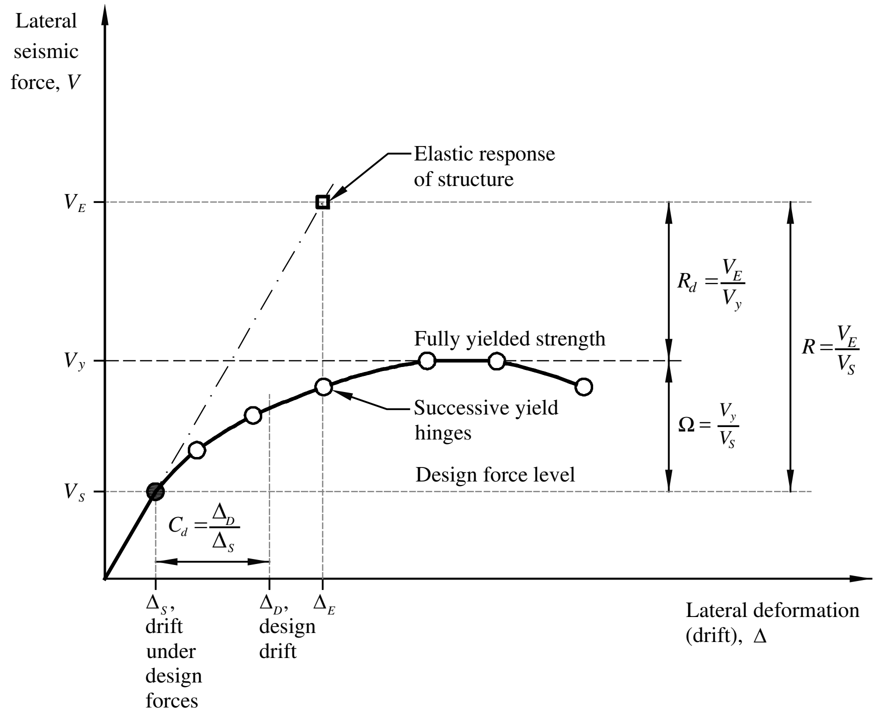

Figure 2.1: Inelastic force-deformation curve (Figure from FEMA P750, FEMA 2009)

In the NBC 2015, two modification factors, i.e., ductility-related force modification factor R_d (equivalent to R_d in ASCE/SEI 7-16) and overstrength-related force modification factor R_o (equivalent to Ω_0), are used to reduce the structure's seismic demand. R is essentially the same as the product of R_d and R_o. Values of R_d and R_o are presented in Table 4.1.8.9 in the NBC 2015.

It is found that R_o in the NBC 2015 varies between 1.3 and 1.7. That is less than Ω_0 in ASCE/SEI 7-16, which ranges from $2\frac{1}{2}$ to 3. Nevertheless, R and $R_d R_o$ values are close for common bearing wall systems, moment-resisting systems, and *building* frame systems.

In the NBC 2015, an additional higher mode factor M_v is included to account for higher mode effect on base shear. M_v values are listed in Table 4.1.8.11 of the NBC 2015 and have been reproduced in Table 2.5. It shows that M_v depends on the *building's* fundamental period T_a, type of SFRS, and ratio $S_a(0.2)/S_a(5.0)$. These ratios, corresponding to *Site Classes* A, B, C, D, and E, are calculated for all 679 locations listed in the NBC 2015. *Site Class F* is not included because the site coefficients for this *Site Class* are unavailable in the NBC 2015. It is found that the maximum ratio is less than 20; therefore, according to Table 2.5, M_v has no effect on the base shear of structures with natural periods $T_a \leq 2.0$ s (except one case marked by †). It is concluded that, for *buildings* with natural periods $T_a \leq 2.0$ s, M_v has negligible effect on the base shear formula in the NBC 2015 (Table 2.4), compared to the formula in ASCE/SEI 7-16. Given the similarities in base shear formulas and seismic provisions in material design standards in Canada and U.S. (Uzumeri et al. 1978; Allen 1975), the *building* capacity and fragility parameter values in FEMA P-154 are adopted to develop *building* capacity and fragility curves for sixteen *model building types* in the *Level 2 – SQST*.

Table 2.5: Higher mode factor M_v in the NBC 2015

$S_a(0.2)/S_a(2.0)$	$T_a \leq 0.5$	$T_a=1.0$	$T_a=2.0$
Moment-resisting frames, coupled walls			
5	1	1	1
20	1	1	1
Braced frames			
5	1	1	1
20	1	1	1
Walls, wall frame systems, other systems			
5	1	1	1
20	1	1	1.18[†]

2.2.4 **Seismic zones**

Seismic zones were widely used in the 20th century to group locations where *seismicity* is fairly similar. A variety of design code levels associated with different seismic design restrictions and detailing requirements were assigned to *buildings* in different *seismic zones*. *Seismic zones* were abandoned and replaced by seismic design categories in recent model *building* codes in the U.S. and Canada. In addition to *seismicity*, the seismic design category also incorporates site condition and the *building* importance category, aiming to achieve acceptable and consistent *seismic risk* in the design of new *buildings*. Since FEMA P-154 developed *building* capacity and fragility curves as functions of *seismic zones*, *seismic zones* are also used in the *Level 2 – SQST*. In the following, *seismic zones* in the U.S. are presented and mapped to Canadian *seismic zones*.

The *seismic zone* map was first incorporated in the 1935 edition of the UBC (i.e. 1935 UBC) for eleven western states in the U.S. Four *seismic zones* – 0, 1, 2, and 3 – were developed based on approximately equal probability. In the 1949 UBC, four qualitative *seismic zones* – 0, 1, 2, and 3 – were defined to relate ground motion intensity to the severity of anticipated *building damage*, i.e., no *damage* (comparable to the Modified Mercalli Intensity, MMI IV or less), minor *damage* (comparable to MMI V and VI), moderate *damage* (comparable to MMI VII), and major *damage* (comparable to MMI VIII or higher). As the MMI increases, higher design code levels regarding more stringent seismic design requirements are enforced. The *buildings* in Zone 1 to Zone 3 correspond to Pre-code, Low-code, and Moderate-code design levels, respectively. In the 1976 UBC, significant improvements in seismic design provisions were made in response to the 1971 San Fernando earthquake. For example, Zone 4 was first introduced to categorize areas within Zone 3 that are close to certain major fault systems (e.g., coastal California). In Zone 4, ordinary *buildings* were assigned the High-code design level, while essential *buildings* were assigned the

Special High-code design level. Other improvements include the introduction of soil profiles, importance factors, seismic requirements for concrete moment frame construction, etc. Therefore, the 1976 UBC is deemed as the first modern *building* code, and higher design levels are chosen for post-1975 *buildings* (FEMA 2012a; 2012b). FEMA P-154 determined post-benchmark modifiers based on the parameter values for post-1975 *buildings* and engineering judgement.

In FEMA P-154, five *seismicity* regions are defined in terms of short-period spectral response acceleration S_{MS} (S_a at 0.2-second period) and long-period spectral response acceleration S_{M1} (S_a at 1-second period) corresponding to Soil Type B: Low, Moderate, Moderately High, High, and Very High. Table 2.6 presents the S_{MS} and S_{M1} threshold values for each *seismicity* region. In determining thresholds for *seismicity* regions, full MCE_R spectral response accelerations are used. Note that site coefficients are not incorporated into the threshold values of spectral response accelerations. This is consistent with the determination of *seismic zones* and corresponding design code levels in the 1997 UBC and its earlier editions.

Table 2.6: Seismicity regions in FEMA P-154

Seismicity region	**MCE_R spectral response acceleration**			
	S_{MS}		S_{M1}	
	>	≤	>	≤
Low		0.25 g		0.10 g
Moderate	0.25 g	0.5 g	0.10 g	0.2 g
Moderately High	0.5 g	1.0 g	0.2 g	0.4 g
High	1.0 g	1.5 g	0.4 g	0.6 g
Very High	1.5 g		0.6 g	

Table 2.7 summarizes mapping between *seismicity* regions in FEMA P-154 and design code levels used in the UBC editions. It is noted that, in pre-1976 UBC editions, the Pre-code design level corresponds to MMI V (comparable to Low *seismicity* region in FEMA P-154) and MMI VI (comparable to Moderate *seismicity* region in FEMA P-154). FEMA P-154 project team made an adjustment for the Moderate *seismicity* region, which is a higher design code level as compared to that assigned to the Low *seismicity* region.

Table 2.7: Mapping between seismicity regions and design code levels in FEMA P-154

Seismicity region	Pre-code design	Pre-benchmark design	Post-benchmark design	MMI scale
Low	Pre-code	Pre-code	Low-code	V
Moderate	Pre-code	Low-code	Moderate-code	VI
Moderately High	Pre-code	Average of Low- and Moderate-code	Average of Moderate- and Special High-code	VII ½
High	Pre-code	Moderate-code	Special High-code	VIII+
Very High	Pre-code	Moderate-code	Special High-code	

In Canada, *seismic zones* were last referenced in the NBC 1995. Beginning with the NBC 2005, *seismic zones* were replaced in favour of the uniform hazard spectrum (UHS) approach for design ground motions with a probability of exceedance of 2% in 50 years (i.e. return period of 2475 years). In *Level 2 – SQST*, *seismic zones* correspond to the site seismic categories for *Site Class* C in the *Level 1 – PST*. *Seismic zones* were preferred over site-specific UHS in absence of available capacity and fragility curves for *buildings* in different specific sites across Canada.

Table 2.8 presents the six *seismic zones* in the *Level 2 – SQST*: Very Low (VL), Low (L), Moderate (M), Moderately High (MH), High (H), and Very High (VH). Similar to FEMA P-154, site condition and importance category are not incorporated into the determination of *seismic zones*.

Table 2.8: *Seismic zones* in *Level 2 – SQST*

Seismic zone	Max[$S_a(0.2)$, $S_a(0.5)$]		$S_a(1.0)$		MMI scale
	>	≤	>	≤	
Very Low		0.10 g		0.05 g	V
Low	0.10 g	0.20 g	0.05 g	0.10 g	VI
Moderate	0.20 g	0.35 g	0.10 g	0.15 g	VI $\frac{1}{2}$
Moderately High	0.35 g	0.75 g	0.15 g	0.30 g	VII $\frac{1}{2}$
High	0.75 g	1.15 g	0.30 g	0.50 g	VIII+
Very High	1.15 g		0.50 g		

Recall that, in the *Level 1 – PST*, site condition is incorporated into seismic categorization. The *Level 1 – PST* assesses the *seismic risk* of *existing buildings* based on a number of key criteria (e.g., *seismicity* and *benchmark NBC edition*). Given that site condition can impact the seismic performance of *existing buildings*, site condition is incorporated into *seismicity* criteria. In *Level 2 – SQST*, structural *seismic risk* is quantitatively determined by calculating the structural basic score and score modifiers. The effect of site condition on a *building's* seismic performance is captured by *Site Class* modifier.

Table 2.9 summarizes the mapping between *seismic zones* in the *Level 2 – SQST* and *seismicity* regions in FEMA P-154. The mapping is based on the MMI values that correspond to different levels of *seismicity*. Given the similarities in the seismic provisions of model *building* codes and referenced material design standards in Canada and the U.S., the design code levels in FEMA P-154 are adopted to determine *building* capacity and fragility parameter values for the different *seismic zones*.

Table 2.9: Mapping between *seismic zones* in the *Level 2 – SQST* and *seismicity* regions in FEMA P-154

<table>
<tr><th>Level 2 – SQST
seismic zone</th><th>FEMA P-154
seismicity region</th><th>MMI scale</th></tr>
<tr><td>Very Low and Low</td><td>Low</td><td>V</td></tr>
<tr><td>Moderate</td><td>Moderate</td><td>VI</td></tr>
<tr><td>Moderately High</td><td>Moderately High</td><td>VII</td></tr>
<tr><td>High</td><td>High</td><td rowspan="2">VIII+</td></tr>
<tr><td>Very High</td><td>Very High</td></tr>
</table>

In the *Level 2 – SQST*, mean values of spectral accelerations at short and long periods are used to determine the seismic demands of *existing buildings* in each *seismic zone*. The mean values are calculated based on the seismic data for 679 locations across Canada in Appendix C of Division B of the NBC 2015. Note that the seismic data in Appendix C are determined assuming *Site Class* C. However, *buildings* in the 679 locations may be situated on *Site Class* A through *Site Class* F, depending on local soil conditions. To investigate the effect that selecting reference *Site Class* (i.e. the basis for calculating structural basic scores) has on the mean values in each *seismic zone*, five sets of mean values are calculated by assuming all locations correspond to *Site Classes* A, B, C, D, and E, respectively. *Site Class F* is not considered because no site coefficients are provided for this *Site Class*. In the *Level 2 – SQST*, presence of *Site Class F* will trigger *Level 3 – SEG*.

The procedure to determine mean values of $S(0.2)$ (greater between $F(0.2)S_a(0.2)$ and $F(0.5)S_a(0.5)$) and $S(1.0)$ (i.e. $F(1.0)S_a(1.0)$), assuming that all locations are situated on *Site Class* C ($F(0.2) = 1.0$, $F(1.0) = 1.0$), is presented as follows: (1) select a location and compare its short-period spectral acceleration (greater between $S_a(0.2)$ and $S_a(0.5)$) and long-period spectral acceleration ($S_a(1.0)$) to the thresholds shown in Table 2.8; (2) categorize *seismic zones* for all 679 locations in terms of the short- and long-period spectral accelerations, respectively; and (3) calculate mean values of short- and long-period spectral accelerations for each *seismic zone*.

The procedures to determine mean values of spectral accelerations corresponding to *Site Classes* A, B, D, and E are the same as the procedure for *Site Class* C, except for the first step, since site coefficients $F(T)$ vary with the reference peak ground acceleration (PGA_{ref}) and fundamental period *T*. Step (1) is thus divided into three sub-steps: (1a) select a location and determine its PGA_{ref}; (1b) calculate short-period spectral acceleration $S(0.2)$ and 1-second period spectral acceleration $S(1.0)$; and (1c) compare $S(0.2)$ and $S(1.0)$ to the thresholds in Table 2.8, respectively.

Table 2.10 provides five sets of mean values of spectral accelerations for all *seismic zones* by assuming all 679 locations are situated on *Site Classes* A, B, C, D, E, respectively. Note that Very High Max (VHX) *seismicity* is added here to show the extreme end of the Very High (VH) *seismic zone*, approximating the maximum *seismicity* in Canada that corresponds to different *Site Classes*. It is shown that *Site Class* has a superficial effect on the mean values of spectral response accelerations $S(0.2)$ and $S(1.0)$ in all *seismic zones* except High and Very High *seismic zones*. This is mainly due to the limited locations in High and Very High *seismic zones*, resulting in large variability when calculating the mean values. For consistency with the NBC 2015, *Site Class* C is considered as the reference *Site Class* for calculating structural basic scores.

Table 2.10: Mean values of spectral response accelerations corresponding to the five Site Classes A, B, C, D, and E

Seismic zone	Site Class A		Site Class B		Site Class C		Site Class D		Site Class E	
	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$
Very Low	0.06	0.03	0.07	0.03	0.07	0.03	0.08	0.03	0.09	0.05
Low	0.14	0.07	0.14	0.07	0.14	0.07	0.15	0.08	0.15	0.07
Moderate	0.28	0.12	0.27	0.13	0.26	0.12	0.27	0.13	0.26	0.13
Moderately High	0.48	0.23	0.49	0.25	0.51	0.20	0.51	0.19	0.51	0.21
High	0.94	0.40	0.97	0.40	0.87	0.41	0.89	0.39	0.95	0.37
Very High	1.19	0.51	1.23	0.54	1.40	0.70	1.37	0.72	1.38	0.79
Very High Max	1.19	0.51	1.33	0.57	1.73	0.90	1.56	1.09	1.61	1.25

It is noticed that FEMA P-154 chose *Soil Type* (same as *Site Class*) CD (the average of *Soil Type* C and *Soil Type* D) for calculating basic scores; median values of S_{MS} and S_{M1} in each *seismic zone* corresponding to other Soil Types are used to calculate Soil Type modifiers. The FEMA P-154 project team believes that this approach is more accurate than always adjusting from stiffer soil to softer soil, as was necessary in the second edition of FEMA 154 (2002), in which the reference Soil Type was B. In addition, FEMA P-154 used median values rather than mean values to calculate the seismic demands of *existing buildings* in each *seismicity* region. Median values in each *seismicity* region are determined by multiplying median values of S_{MS} and S_{M1}, assuming Soil Type B, by the site coefficients for short and long fundamental periods. The rationale behind this is unclear because site coefficients should be applied directly to spectral accelerations at specific locations rather than the medium values for all locations within the *seismicity* region.

2.3 *Level 2 – SQST* methodology: structural scoring

The structural scoring methodology quantitatively evaluates the probabilities of collapse of *existing buildings* based on generic *building* capacity and fragility curves that are developed for different *model building types*. A structural score is defined as the common logarithm of probability of collapse for a given *model building type* and *seismic zone*, which is calculated as the sum of a structural basic score and applicable score modifiers. This section presents a brief introduction of the key steps in this structural scoring methodology.

2.3.1 Construction of building capacity curve

The *building* capacity curve is a plot of the base shear and roof displacement from a nonlinear pushover curve that has been converted to acceleration-displacement response spectrum (ADRS) format. The *building* is assumed to be perfectly elastic prior to yielding, and perfectly plastic beyond the ultimate capacity. The transition from yield point to ultimate point of the capacity curve is assumed to be elliptical. Therefore, the entire capacity curve can be constructed based on two control points (D_y, A_y) and (D_u, A_u). In the structural scoring methodology, the *building* capacity curve control points are calculated according to the FEMA P-154 (2015) approach, with modifications to account for *building* importance, *deterioration* and age effect:

$$A_y = k_d \cdot \frac{(I_E C_S)\gamma}{\alpha_1} \tag{2.4}$$

$$D_y = 9.8 A_y T_e^2 \tag{2.5}$$

$$A_u = \lambda A_y \tag{2.6}$$

$$D_u = \lambda \mu D_y \tag{2.7}$$

where *deterioration* and age factor k_d is used to reduce *building* capacity due to *deterioration* and age effect (determination of k_d is presented later in Section 2.5.6). C_s is the seismic design coefficient, which is a fraction of the *building* weight to seismic base shear associated with the design code level. Importance factor I_E is applied to C_s to account for the increase (or decrease) in design base shear as a function of importance category. α_1 is the modal weight factor, which is the fraction of *building* weight participating in the pushover mode. T_e is the fundamental elastic period of the *building*. γ is the yield strength factor, which is the ratio of nominal yield strength to design strength. λ is the ultimate strength factor, which is the ratio of nominal ultimate strength to yield strength. μ is the ductility factor, which is the ratio of ultimate displacement to λ times the yield displacement. Except for k_d and I_E, the parameter values in the above equations are provided in APPENDIX D.

2.3.2 Determination of seismic demand spectrum

In the NBC 2015, 5%-damped spectral accelerations at 0.2 s, 0.5 s, 1 s, 2 s, 5 s, and 10 s are used to construct elastic response spectra for specific locations. However, the *seismic zones* in the *Level 2 – SQST* are defined based on spectral response accelerations at short periods, $S(0.2)$, and at 1-second period, $S(1.0)$. The variations in the spectral shape and amplitude of elastic response spectrum are considered in developing fragility curves.

Similar to FEMA P-154, a standard spectral shape of design response spectra is defined based on $S(0.2)$ and $S(1.0)$, where

$$S(0.2) = \max\begin{cases} F(0.2)S_a(0.2) \\ F(0.5)S_a(0.5) \end{cases} \tag{2.8}$$

$$S(1.0) = F(1.0)S_a(1.0) \tag{2.9}$$

Mean values of $S(0.2)$ and $S(1.0)$, which correspond to reference *Site Class* C, are used to calculate structural basic scores (Table 2.10). Mean values of $S(0.2)$ and $S(1.0)$, which correspond to other *Site Classes*, are determined in Section 2.5.4 and are used to compute the *Site Class* modifiers.

Since elastic damping ratios β_e for certain *model building types* are not equal to 5% (see Table D.2 in APPENDIX D), damping correction factors (provided in APPENDIX C) are used to convert 5%-damped $S(0.2)$ and $S(1.0)$ to β_e-damped $S(0.2)$ and $S(1.0)$. It is noteworthy that the β_e has considered the damping effect of the entire *building,* including structural and non-structural components. Therefore, the effect of energy dissipation from non-structural components is not double-counted in the determination of elastic response spectrum.

The effective damping β_{eff} considers the inelastic energy dissipation and is determined as follows:

$$\beta_{eff} = \beta_e + \beta_H \tag{2.10}$$

$$\beta_H = \kappa_d\left(\frac{Area}{2\pi DA}\right) \tag{2.11}$$

where β_H is the hysteretic damping, D is the spectral displacement of the structure (i.e. S_d in Figure 2.2), A is the spectral response acceleration at spectral displacement (S_a in Figure 2.2), $Area$ is the area enclosed by an idealized hysteretic loop constructed between the positive and negative spectral displacements $\pm S_d$, and κ_d is a degradation factor that defines the amount of hysteretic damping β_H as a function of earthquake duration.

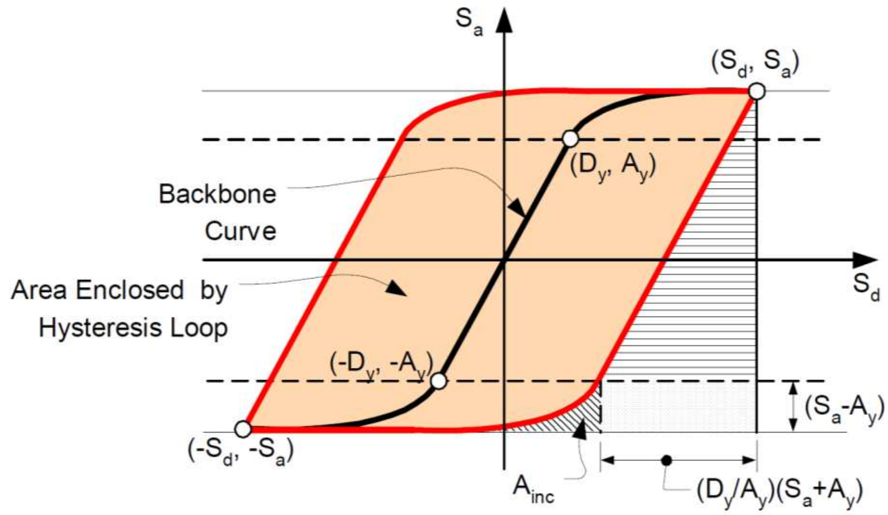

Figure 2.2: Idealized hysteretic loop used to calculate hysteretic damping β_H. (from Tokas and Lobo 2009)

The hysteretic loop by Tokas and Lobo (2009) is used to calculate $Area$. To do this, incremental area A_{inc} is required to be determined for the response in the elliptical region (Figure 2.2). Tokas and Lobo (2009) recommended calculating incremental area A_{inc} numerically. The theoretical solution of A_{inc}, however, is available by analytical integration. In addition, for response in the post-yield plateau corresponding to inelastic response, Tokas and Lobo (2009) calculated incremental area A_{inc} with the following equation:

$$
\begin{aligned}
A_{inc} &= \left(\frac{4-\pi}{2}\right)\left(A_u - A_y\right)\left(D_u - D_y\right) \\
&= 2 \times \left[\left(A_u - A_y\right)\left(D_u - D_y\right) - \frac{\pi}{4}\left(A_u - A_y\right)\left(D_u - D_y\right)\right]
\end{aligned}
\tag{2.12}
$$

The term $\left(A_u - A_y\right)\left(D_u - D_y\right)$ is the area of the rectangle between yield capacity point (D_y, A_y) and ultimate capacity point (D_u, A_u), and the term $\frac{\pi}{4}\left(A_u - A_y\right)\left(D_u - D_y\right)$ represents a quarter of the area enclosed by the entire elliptical shape in Figure 2.2. The equation is based on the assumption that the yield capacity point is the left end of the elliptical shape function. However, this assumption is not true because the tangent line at yield capacity point is not perpendicular to the horizontal axis.

In the structural scoring methodology, $Area$ is calculated by:

For the response in the elastic region, $0 < S_d \le D_y$

$$
S_a = \left(\frac{A_y}{D_y}\right) S_d \tag{2.13}
$$

$A_{inc} = 0$

$Area = 0$

For the response in the elastic-plastic transition phase, $D_y < S_d \leq D_u$ (see APPENDIX E)

$$S_a = b\sqrt{1 - \frac{(S_d - D_u)^2}{a^2}} + k$$

$$A_{inc} = (S_a - k)(S_d - D_y) - \frac{1}{2}ab(t_2 - t_1) - \frac{1}{4}ab[\sin(2t_2) - \sin(2t_1)] \tag{2.14}$$

$$t_1 = \sin^{-1}[(D_y - D_u)/a],\ t_2 = \sin^{-1}[(S_d - D_u)/a]$$

$$Area = 4S_aS_d - (S_a + A_y)^2\left(\frac{D_y}{A_y}\right) - 2(S_a^2 - A_y^2)\left(\frac{D_y}{A_y}\right) - 2A_{inc}$$

where *a*, *b* ,and *k* are obtained from Eqns. (C.11) through (C.13).

For the response in the post-yield plateau, $S_d > D_u$ (see APPENDIX E)

$$S_a = A_u$$

$$A_{inc} = (A_u - k)(D_u - D_y) + \frac{1}{2}abt_1 + \frac{1}{4}ab\sin(2t_1)$$

$$t_1 = \sin^{-1}[(D_y - D_u)/a] \tag{2.15}$$

$$Area = 4S_aS_d - (S_a + A_y)^2\left(\frac{D_y}{A_y}\right) - 2(S_a^2 - A_y^2)\left(\frac{D_y}{A_y}\right) - 2A_{inc}$$

2.3.3 Calculation of peak spectral displacement

Peak spectral displacement δ_p (also called performance point) is defined as the intersection of the *building* capacity curve and the locus demand spectrum, implying a sense of dynamic equilibrium (FEMA 440 2005). Peak spectral displacement is the expected *building* displacement given the code level earthquake (CLE) corresponding to a specified *seismic zone*. A recursive solution technique for determining δ_p has been developed as part of the *Level 2 – SQST*. The basis of the technique stems from the observation that δ_p is a function of β_H, which itself is dependent on δ_p. Therefore, the performance point can be recursively determined as follows:

1. Select an arbitrary initial spectral displacement D_i (for example, select D_y);
2. Compute the corresponding spectral response acceleration A_i from the capacity curve;

3. Compute the period $T_i = \sqrt{D_i/9.8A_i}$;
4. Calculate hysteretic damping β_H (Eq. (2.11)) to obtain the area enclosed by the hysteretic loop (i.e. $Area$);
5. Calculate effective damping β_{eff} (Eq. (2.10));
6. Calculate damping reduction factors R_A and R_v (Eqns. (C.18) and (C.19));
7. Calculate β_{eff}-damped spectral acceleration $S_a(T_i)$;
8. Find spectral displacement D_{i+1} using T_i and $S_a(T_i)$ (Eq. (C.20));
9. Calculate convergence parameter $r = \left|\frac{D_{i+1}-D_i}{D_{i+1}}\right|$;
   - If $r < 1\%$, convergence has been achieved and performance point $\delta_p = D_{i+1}$;
   - If $r > 1\%$, set $D_i = D_{i+1}$ and return to Step 1.

The recursive solution generally converged within five iterations. This approach is more efficient than the FEMA P-154 graphic approach, which determines peak spectral displacement using an iterative procedure that requires considerable effort.

2.3.4 **Development of fragility curves**

The structural complete *damage* state (FEMA 2012a; 2012b) is used for assessing *existing buildings* with respect to a specified importance category, such as *Low*, *Normal*, *High*, and *Post-disaster*. Given any value of spectral response, e.g., spectral displacement S_d for the structure, probability of a structure being in complete *damage* state is determined by:

$$P[Complete\ damage|S_d] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{S_d}{S_{d,m}}\right)\right] \tag{2.16}$$

where $S_{d,m}$ is the median value of spectral displacement corresponding to the threshold of the structural complete *damage* state, and β_d is the logarithmic standard deviation of the overall variability associated with the structural complete *damage* state.

$S_{d,m}$ is calculated using Eq. (C.27), with modifications to account for *building* importance category:

$$S_{d,C} = k_\Delta\, \Delta_c\, H_R\, (\alpha_2/\alpha_3) \tag{2.17}$$

where k_Δ is a factor used to increase or decrease a *building's* interstorey drift capacity with an importance category other than *Normal importance category*. This will be discussed in Section 2.5.3. Δ_c is the inter*storey* drift ratio corresponding to structural complete *damage* state, H_R is the *building* height at roof level in inches, and α_2 and α_3 are modal height factor and modal shape factor relating maximum-*storey* drift and roof drift. The drift parameter values in Eq. (2.18) are provided in APPENDIX D.

Standard deviation β_d associated with the structural complete *damage* state is determined using Eq. (2.18):

$$\beta_d = \sqrt{\frac{\beta_{C,D}^2 - (\beta_{CAP}^2 + \beta_{T,C}^2)}{X\left(1 + \frac{D_e}{D}\right)} + (\beta_{CAP}^2 + \beta_{T,C}^2)} \tag{2.18}$$

where $\beta_{C,D}$ represents standard deviation that is deterministicly based on HAZUS AEBM (2012a) and OSHPD HAZUS (2016), which depends on standard deviation β_{CAP} of capacity variability and $\beta_{T,C}$ of *damage* state threshold variability. D_e and D are elastic and peak spectral displacements, respectively. X is an adjustment factor.

For normal importance *buildings*, $\beta_{CAP} = 0.3$, $\beta_{T,C} = 0.4$, and $X = 0.75$ are used in developing fragility curves associated with the complete damage state of generic *model building types*, which are consistent with FEMA P-154. Since the structural scoring methodology incorporates importance category into the scoring process, $\beta_{C,D}$, β_{CAP} , and $\beta_{T,C}$ values for importance categories other than *Normal importance* are proposed and discussed in Section 2.5.3.

Having determined $S_{d,m}$ and β_d, the probability of a structure being in complete *damage* state given spectral displacement S_d can be determined using Eq. (2.16) by changing the values of S_d from a small displacement such as $D_u/500$ to a large displacement S_d>D_u.

Given peak spectral displacement δ_p obtained in Section 2.3.3, the probability of a structure being in the complete *damage* state is determined by:

$$P\left[Complete\ damage\middle|\delta_p\right] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{\delta_p}{S_{d,m}}\right)\right] \tag{2.19}$$

2.3.5 Calculation of structural basic scores

An empirical collapse factor CF, i.e. the expected value of collapsed area, is used to associate the probability of a structure being in complete *damage* state to probability of collapse:

$$P(COL|CLE) = P\left[Complete\ damage\middle|\delta_p\right] \times CF \tag{2.20}$$

where CF is dependent on *model building type* and type of *building irregularity* (see APPENDIX D). *CLE* refers to code level earthquake assuming *Site Class* C. CLE is determined in accordance with Section 2.3.2.

Having obtained $P(COL|CLE)$, structural basic score S_B is computed by:

$$S_B = -\log_{10}\big(P(COL|CLE)\big) \tag{2.21}$$

S_B represents the expected structural score for a generic *model building type* corresponding to a specified *seismic zone*. Higher S_B value indicates lower probability of collapse.

2.3.6 Calculation of structural score modifiers

Presence of certain *building* conditions in specific *buildings* can jeopardize a *building's* seismic performance in a number of different ways. For example, vertical *irregularity* can reduce drift capacity and increase collapse factor. As a result, probability of collapse increases while the structural score decreases. However, beneficial conditions such as post-benchmark design could reduce the probability of collapse and thus increase the structural score. In order to address the effects of *building* conditions on the seismic performance of specific *buildings*, structural score modifiers are determined for applicable *building* conditions.

Given a specified *building* condition *i*, the modified structural score $S_{Condition\ i}$ is calculated as:

$$S_{Condition\ i} = -\log_{10}(P[COL|CLE, Condition\ i]) \tag{2.22}$$

where $P[COL|CLE, Condition\ i]$ is the probability of collapse of the *building* given the *CLE* and condition *i*. It is calculated by modifying parameter values regarding condition *i* and keeping other parameter values unchanged. The adjustment is based on the same manner as FEMA P-154 for its Level 1 score modifiers.

Finally, score modifier M_i for condition *i* is determined by:

$$M_i = S_{Condition\ i} - S_B \tag{2.23}$$

A positive M_i value indicates that the condition *i* has a beneficial effect on the *building's* seismic performance, while a negative M_i value indicates that the condition *i* has a detrimental effect on the *building's* seismic performance.

2.3.7 Determination of structural score

Having obtained the structural basic score S_B and applicable score modifiers M_i, the structural score *S* is calculated as the sum of the basic score and applicable score modifiers:

$$S = S_B + \sum_i M_i \tag{2.24}$$

The summation is an approximate approach disregarding the potential interconnections among *building* conditions, which could result in negative structural scores. However, a negative score does not have physical meaning because it indicates a probability of collapse greater than 100%. To avoid possible negative scores, a minimum score $S_{\min}$ is developed for each *model building* *type* in each *seismic zone* by assuming pre-code design, moderate *building deterioration*, *severe vertical irregularity*, and plan *irregularity,* situated on *Site Class* E. *Building* importance was not incorporated into the determination because the importance category is not applied to *pre-code buildings*.

If the structural score based on Eq. (2.25) is less than the minimum score, the structural score is taken as the minimum score, i.e.

$$S = S_B + \sum_i M_i \geq S_{\min} \tag{2.25}$$

2.4 Structural basic scores

Structural basic scores (S_B) are calculated for each of the sixteen *model building types* in each *seismic zone* and presented in the *Level 2 – SQST* screening forms for each *seismic zone*. Note that the *Level 2 – SQST* screening form is not provided for Very High Max *seismicity* because the Very High Max corresponds to the extreme end of the Very High *seismic zone*. Nevertheless, S_B values for Very High Max *seismicity* (Table 2.10) are calculated in this section.

The S_B value for a given *model building type* and a *seismic zone* is calculated based on the following assumptions:

- No *building irregularity*;
- Pre-benchmark design period (between the *pre-code NBC edition* and applicable *benchmark NBC edition*);
- *Normal importance* category;
- *Site Class* C;
- Low-rise *building* (1~3 *storeys*);
- No to negligible *building deterioration*;
- Insufficient *redundancy*;
- No *pounding* potential between adjacent *buildings*;
- *Remaining occupancy time* greater than 10 years; and
- No comprehensive *seismic upgrading*.

2.4.1 Seismic zones

Six *seismic zones* were developed based on Canadian *seismicity*. The seismic zones reflect changes in earthquake design according to the NBC 2015 (see Table 2.8). Mean values of short- and long-period spectral accelerations in each *sesismic zone* (see Table 2.10) are used to calculate structural basic scores.

2.4.2 Model building types

Sixteen *model building types*, WLF, WPB, SMF, SBF, SLF, SCW, SIW, CMF, CSW, CIW, PCW, PCF, RML, RMC, URM, and MH are considered (see Table 2.1).

2.4.3 Parameter values for calculating structural basic scores

The specific parameters necessary to construct capacity curves (i.e., H_R, T_e, C_s, γ, λ, μ and α_1), seismic demand spectra (i.e., β_e and κ_d), fragility curves (i.e., Δ_C, α_2, α_3, $\beta_{C,D}$ and β_d), and *building* collapse factor (i.e., CF), were provided in APPENDIX D. Although WLF is mapped to W2 (Table 2.1), parameter values for W1 in FEMA P-154 are used to calculate structural basic scores for WLF. In FEMA P-154, structural basic scores for W1 are higher than those for W2, indicating that W1 generally performs better than W2. According to Doudak (personal communication, March 5, 2020), WLF *buildings* generally perform better than non-engineered WLF *buildings* under Part 9 of the NBC (equivalent to W1 *buildings*). This statement indicates that using W2 parameter values will result in too conservative estimates of structural basic scores for WLF. Given this reason, W1 parameter values are used to calculate structural basic scores for WLF.

2.4.4 Sample calculation of structural basic score

This section presents a sample structural basic score (S_B) calculation for the steel moment frame (SMF) *model building type* in a Moderately High (MH) *seismic zone,* to illustrate the general procedure for calculating S_B values for sixteen *model building types* in all *seismic zones*. The calculations are in imperial units because parameter values in APPENDIX D are adopted from FEMA P-154, where imperial units are used. As a result, the displacements presented in the example are calculated in inches and converted to mm (provided within parentheses).

***Step 1*: Develop building capacity curve**

The capacity curve for a one-*storey* SMF *model building type* in the MH *seismic zone* is determined. The *building* is assumed to be perfectly elastic prior to yielding and perfectly plastic beyond the ultimate capacity. The transition from yield point to ultimate point of the capacity curve is assumed to be elliptical. Therefore, the entire capacity curve can be constructed based on two control points (D_y, A_y) and (D_u, A_u). Capacity curve parameter values are provided in Table D.1 in APPENDIX D. It is noted that $I_E = 1.0$ (*Normal importance*) and $k_d = 1.0$ (*building* without *deterioration* and age effect) are used in calculating S_B. The *building's* yield and ultimate capacities are calculated as:

$$A_y = k_d \cdot \frac{(I_E C_s)\gamma}{\alpha_1} = \frac{1.0 \times 1.0 \times 0.082 \times 2.7}{0.75} = 0.295 \text{ g}$$

$$D_y = 9.8 A_y T_e^2 = 9.8 \times 0.295 \times 0.4^2 = 0.463 \text{ in (11.8 mm)}$$

$$A_u = \lambda A_y = 2 \times 0.295 = 0.59 \text{ g}$$

$$D_u = \lambda \mu D_y = 2 \times 6 \times 0.463 = 5.556 \text{ in (141.1 mm).}$$

The parameter values associated with the elliptical shape in the transition segment of capacity curve are obtained as

$$a = b\sqrt{\frac{D_y}{A_y}\frac{\left(D_u - D_y\right)}{\left(A_y - k\right)}} = 5.117$$

$$b = A_u - k = 0.328$$

$$k = \frac{A_u^2 - A_y^2 + \frac{A_y^2}{D_y}\left(D_y - D_u\right)}{2\left(A_u - A_y\right) + \frac{A_y}{D_y}\left(D_y - D_u\right)} = 0.262$$

The value A between the yield and ultimate control points of the capacity curve is determined by

$$A = b\sqrt{1 - \frac{(D - D_u)^2}{a^2}} + k$$

Changing the value of D from 0 to 6 in. (a larger upper limit could be established when needed) results in the capacity curve for a one-*storey* SMF in the MH *seismic zone* shown in Figure 2.3.

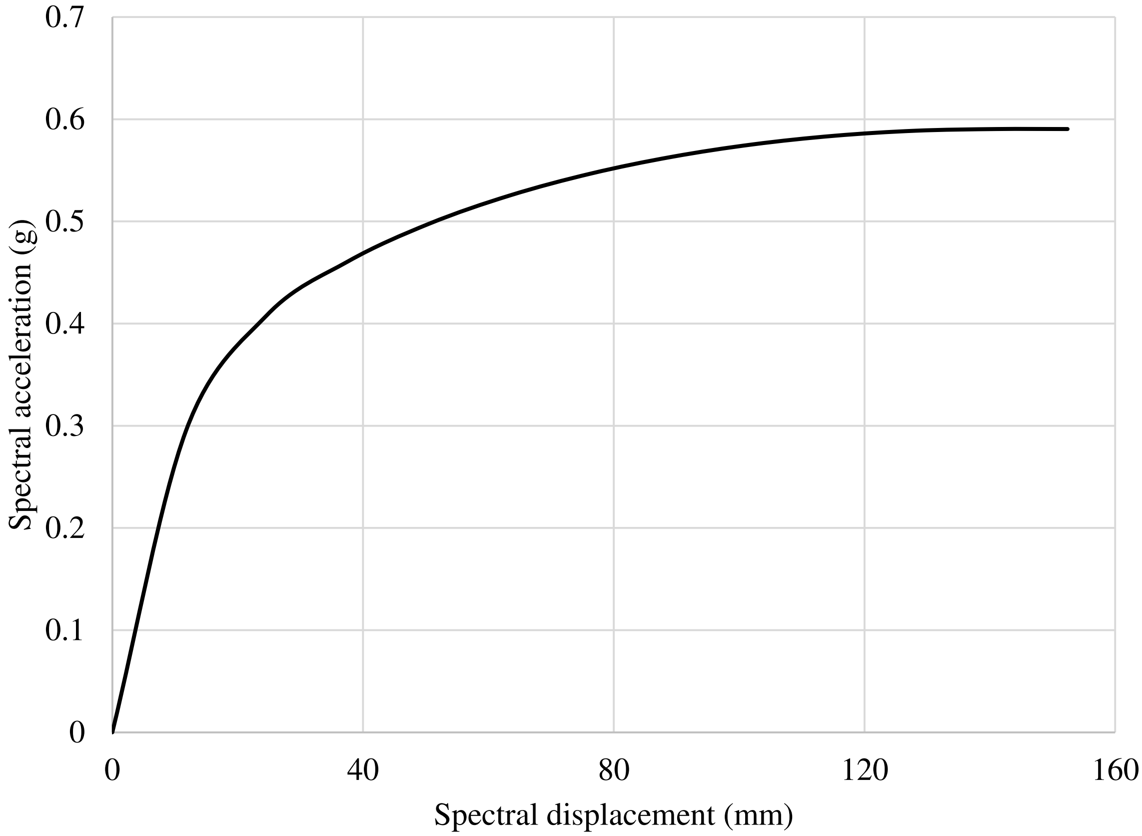

Figure 2.3: Capacity curve for a one-storey SMF building in the MH seismic zone

***Step 2*: Construct seismic demand spectrum**

The 5%-damped ($\beta_e = 5\%$ for SMF) elastic response spectrum is determined first. Spectral response accelerations at short periods, max[S_a(0.2),S_a(0.5)]=0.51 g, and at 1-second period $S_a(1.0) = 0.2$ g are obtained from Table 2.10. Spectral response accelerations of the elastic response spectrum are then calculated by:

$S_a(T) = \max[S_a(0.2), S_a(0.5)]/R_A$ for $0 \leq T \leq T_s$

$S_a(T) = (S_\mathrm{a}(1.0)/T\,)/R_V$ for $T_S < T \leq T_{VD}$

$$T_S = \left[\frac{S_\mathrm{a}(1.0)}{\max[S_a(0.2), S_a(0.5)]}\right] \cdot \left(\frac{R_A}{R_V}\right)$$

$$T_{VD} = 10^{\frac{M_W - 5}{2}}$$

The spectral displacement of the elastic response spectrum is given by:

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2$$

Recall that $R_A = 1.0$ and $R_V = 1.0$ for elastic damping ratio β_e equal to 5%. $M_w = 7.0$ is adopted from FEMA P-154. $T_S = 0.38$ sec. and $T_{VD} = 10$ sec. are thus obtained. Changing the value of period T from 0 to 10 seconds would obtain the 5%-damped elastic response spectrum shown in Figure 2.4.

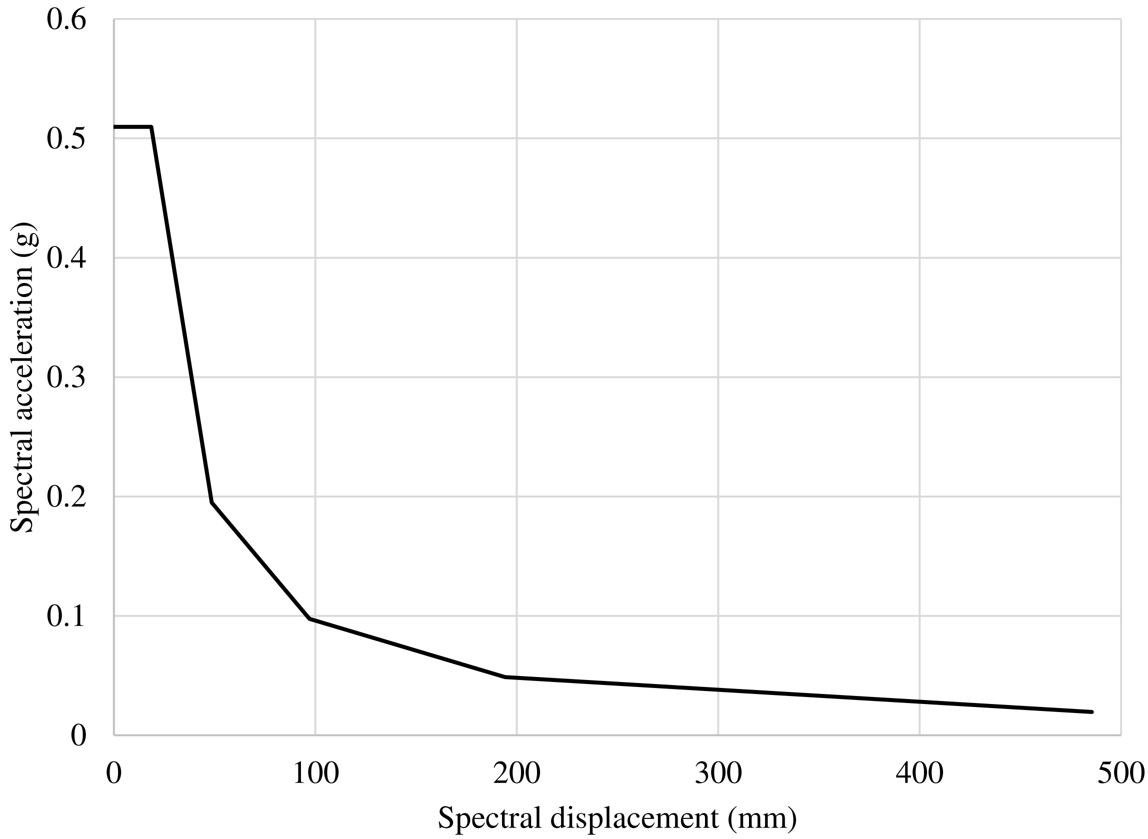

Figure 2.4: 5%-damped elastic response spectrum for a one-storey SMF in the MH *seismic zone*

Based on Table D.1 in APPENDIX D, the natural period T_e for a one-*storey* SMF is equal to 0.4 sec., which is slightly greater than T_S. Therefore, spectral acceleration and spectral displacement for this *building* in the MH *seismic zone* are calculated by:

$$S_a(T) = (S_a(1.0)/T)/R_V = (0.2/0.4)/1.0 = 0.5 \text{ g}$$

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 = 9.8 \times 0.5 \times 0.4^2 = 0.784 \text{ in} = D_e$$

The seismic demand spectrum for a one-*storey* SMF in the MH *seismic zone* is determined based on the procedure presented in Section 2.3.2. Starting at a small displacement of $D_u/500$, a series of 500 points on the β_{eff}-damped seismic demand spectrum are calculated. Sample calculations are presented herein for four of those points.

Point 1: At start of capacity curve, the *building* is within the elastic region

$D = \frac{D_u}{500} = 0.011$ in (0.28 mm)

$$A = \frac{D}{D_y} \times A_y = 0.007 \text{ g}$$

$$T = \sqrt{\frac{D}{9.8 \times A}} = 0.4 \text{ s}$$

$Area = 0$ (still on elastic portion of demand spectrum)

$\beta_H = 0$

$\beta_{eff} = \beta_e + \beta_H = 5\%$

$$R_A = \frac{2.12}{3.21 - 0.68 \ln \beta_{eff}} = 1.0$$

$$R_V = \frac{1.65}{2.31 - 0.41 \ln \beta_{eff}} = 1.0$$

$S_a(T) = (S_a(1.0)/T)/R_V = (0.2/0.4)/1.0 = 0.5$ g

$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 = 9.8 \times 0.5 \times 0.4^2 = 0.784$ in (19.9 mm)

Therefore, seismic demand spectrum is the same as elastic response spectrum at point 1. Repeat the procedure with small increments of D until one gets to:

Point 21: At yield point on capacity curve, the *building* is at the elastic capacity point

$D = D_y = 0.463$ in
$A = A_y = 0.295$ g

$$T = \sqrt{\frac{D}{9.8 \times A}} = 0.4 \text{ s}$$

$Area = 0$ (still on elastic portion of the capacity curve)

$\beta_H = 0$

$\beta_{eff} = \beta_e + \beta_H = 5\%$

$$R_A = \frac{2.12}{3.21 - 0.68 \ln \beta_{eff}} = 1.0$$

$$R_V = \frac{1.65}{2.31 - 0.41 \ln \beta_{eff}} = 1.0$$

$$S_a(T) = (S_a(1.0)/T)/R_V = (0.2/0.4)/1.0 = 0.5 \text{ g}$$

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 = 9.8 \times 0.5 \times 0.4^2 = 0.784 \text{ in}$$

The seismic demand spectrum is still the same as elastic response spectrum. Points 1 through 21 all occur within the elastic portion of the capacity curve. In this range, the period, T, is always equal to the elastic period, T_e, and the effective damping, β_{eff}, is always equal to the elastic damping, β_e. Hence, these points all occur at $S_a(T) = 0.5$ g and $S_d(T) = 0.784$ in. Point 1 is equal to Point 21. While the elastic response spectrum as shown in Figure 2.4 starts at zero displacement, the seismic demand spectrum begins at a point, with respect to the *building's* elastic period ($T_e = 0.4$ sec.), in the elastic response spectrum.

Repeat the procedure with small increments of D until one gets to:

Point 101: At halfway point to $\mathbf{D_u}$, the *building* is within the elastic-plastic (elliptical) region

$$D = D_u/2 = 2.778 \text{ in (70.6 mm)}$$

$$A = b\sqrt{1 - \frac{(D - D_u)^2}{a^2}} + k = 0.537 \text{ g}$$

$$T = \sqrt{\frac{D}{9.8 \times A}} = \sqrt{\frac{2.778}{9.8 \times 0.537}} = 0.727 \text{ s}$$

$$t_1 = \sin^{-1}\left[(D_y - D_u)/a\right] = -1.474, \; t_2 = \sin^{-1}[(S_d - D_u)/a] = -0.574$$

$$A_{inc} = (S_a - k)(S_d - D_y) - \frac{1}{2}ab(t_2 - t_1) - \frac{1}{4}ab[\sin(2t_2) - \sin(2t_1)] = 0.184 \text{ in g (4.7 mm g)}$$

$$Area = 4S_aS_d - (S_a + A_y)^2\left(\frac{D_y}{A_y}\right) - 2(S_a^2 - A_y^2)\left(\frac{D_y}{A_y}\right) - 2A_{inc} = 3.88 \text{ in g (98.6 mm g)}$$

$$\beta_H = \kappa_d\left(\frac{Area}{2\pi DA}\right) = 0.4 \times \frac{3.88}{2 \times 3.14 \times 2.778 \times 0.537} = 16.6\%$$

$$\beta_{eff} = \beta_e + \beta_H = 5\% + 16.6\% = 21.6\%$$

$$R_A = \frac{2.12}{3.21 - 0.68 \ln \beta_{eff}} = 1.89$$

$$R_V = \frac{1.65}{2.31 - 0.41 \ln \beta_{eff}} = 1.57$$

$$S_a(T) = (S_a(1.0)/T)/R_V = (0.2/0.727)/1.57 = 0.175 \text{ g}$$

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 = 9.8 \times 0.175 \times 0.727^2 = 0.907 \text{ in (23 mm)}$$

The effective damping (21.6%) has significantly increased compared to elastic damping (5%), which results in the remarkable decrease of spectral acceleration and a significant increase in the *building* period. The spectral displacement has a small increase compared to elastic deformation, D_e. The increase results from the combination effect of spectral acceleration and the *building* period.

Repeat the procedure with small increments of D until one gets to:

Point 151: At ultimate point on the capacity curve, the *building* is within the post-yield plateau

$D = D_u = 5.556$ in (141.1 mm)
$A = A_u = 0.59$ g

$$T = \sqrt{\frac{D}{9.8 \times A}} = \sqrt{\frac{5.556}{9.8 \times 0.59}} = 0.98 \text{ s}$$

$$t_1 = \sin^{-1}\left[(D_y - D_u)/a\right] = -1.474$$

$$A_{inc} = (A_u - k)\left(D_u - D_y\right) + \tfrac{1}{2}abt_1 + \tfrac{1}{4}ab\sin(2t_1) = 0.349 \text{ in g (8.9 mm g)}$$

$$Area = 4S_aS_d - \left(S_a + A_y\right)^2\left(\tfrac{D_y}{A_y}\right) - 2\left(S_a^2 - A_y^2\right)\left(\tfrac{D_y}{A_y}\right) - 2A_{inc} = 10.362 \text{ in g (263.2 mm g)}$$

$$\beta_H = \kappa\left(\frac{Area}{2\pi DA}\right) = 0.4 \times \frac{10.362}{2 \times 3.14 \times 5.556 \times 0.59} = 20.1\%$$

$$\beta_{eff} = \beta_e + \beta_H = 5\% + 20.1\% = 25.1\%$$

$$R_A = \frac{2.12}{3.21 - 0.68\ln\beta_{eff}} = 2.08$$

$$R_V = \frac{1.65}{2.31 - 0.41\ln\beta_{eff}} = 1.67$$

$$S_a(T) = (S_a(1.0)/T)/R_V = (0.2/0.98)/1.67 = 0.12 \text{ g}$$

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 = 9.8 \times 0.12 \times 0.980^2 = 1.13 \text{ in (28.7 mm)}$$

Continue to repeat the procedure with small increments of D. The locus demand spectrum is finally obtained and is shown in Figure 2.5.

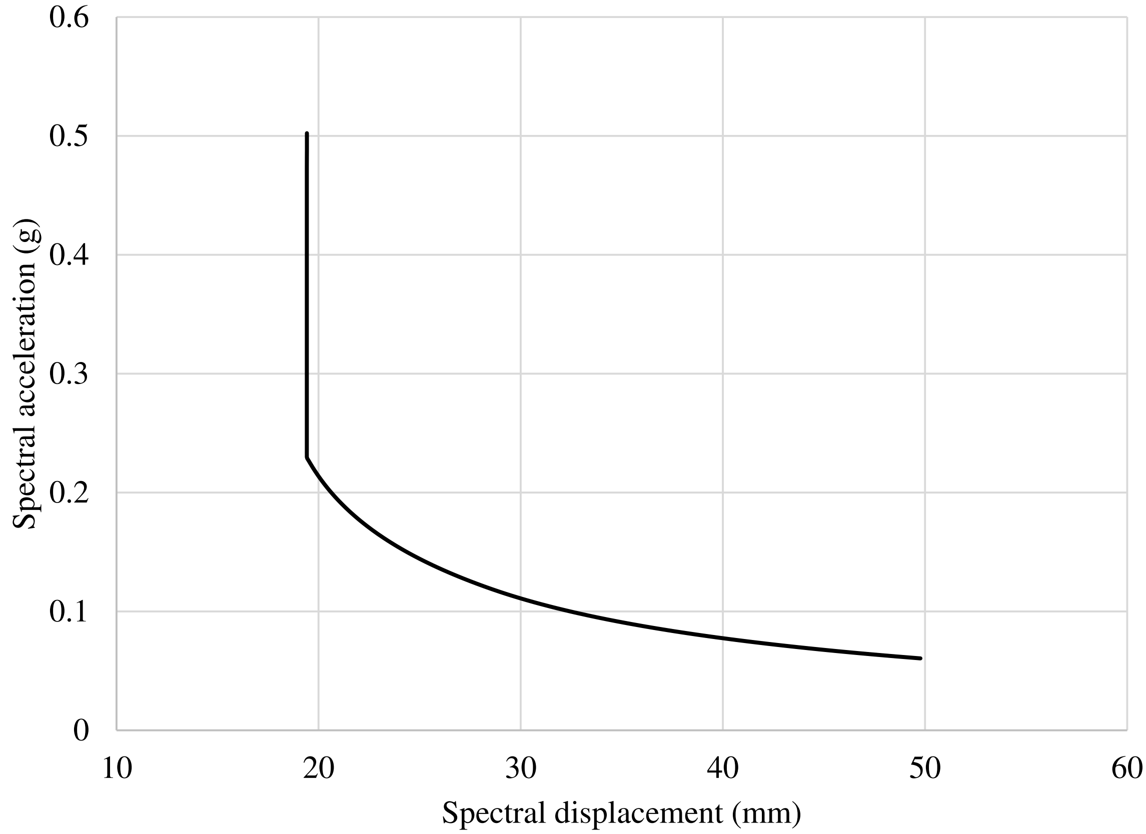

Figure 2.5: β$_{eff}$-% damped locus demand spectrum for a one-storey SMF in the MH *seismic zone*

***Step 3*: Determine peak spectral displacement**

The recursive technique as stated in Section 2.3.3 is used to determine the peak spectral displacement and the result is shown in Figure 2.6. Peak spectral displacement δ_p is equal to D_e, i.e., 0.784 in (19.9 mm).

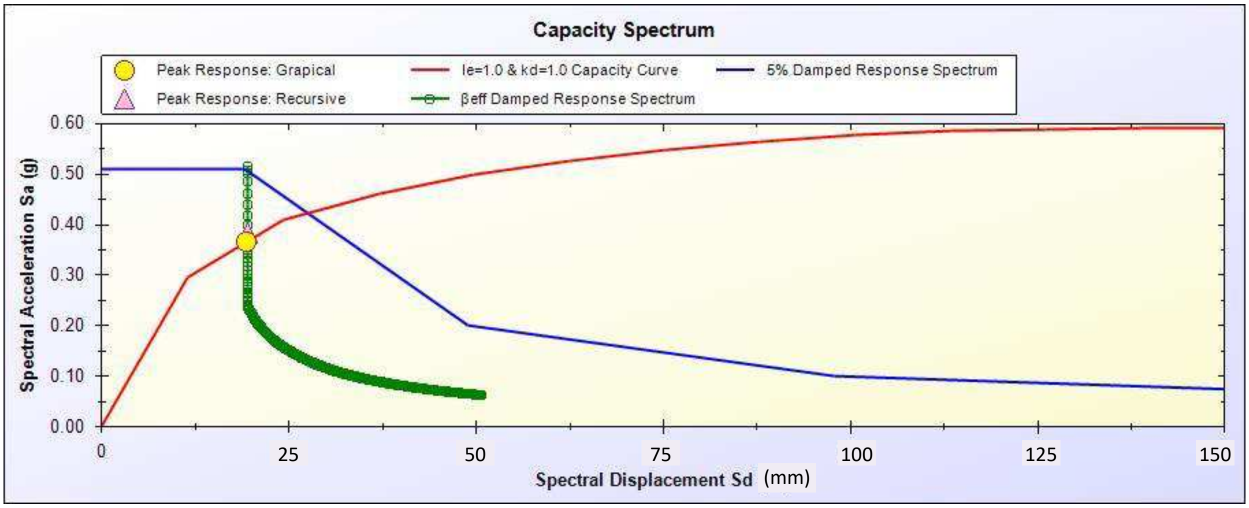

Figure 2.6: Capacity curve, elastic response spectrum, and locus demand spectrum of one-storey SMF in MH *seismic zone*

***Step 4*: Develop fragility curve**

The median spectral displacement with respect to complete *damage* state for a one-*storey* SMF is calculated as:

$$S_{d,m} = k_{\Delta}\,\Delta_c\,H_R\,(\alpha_2/\alpha_3) = 1.00 \times 0.055 \times 14 \times 12 \times \frac{0.75}{1} = 6.93 \text{ in. } (176 \text{ mm})$$

where values of Δ_c, H_R, α_2 and α_3 are obtained from Table D.4, Table D.1 and Table D.5 in APPENDIX D. $k_{\Delta} = 1.0$ is taken for *Normal importance* ($I_E = 1.0$) *buildings* without *deterioration* and age effect.

Logarithmic standard deviation β_d associated with structural complete *damage* state is determined by:

$$\beta_d = \sqrt{\frac{\beta_{C,D}^2 - \left(\beta_{CAP}^2 + \beta_{T,C}^2\right)}{X\left(1 + \frac{D_e}{D}\right)} + \left(\beta_{CAP}^2 + \beta_{T,C}^2\right)} = \sqrt{\frac{0.95^2 - (0.3^2 + 0.4^2)}{0.75\left(1 + \frac{0.784}{0.784}\right)} + (0.3^2 + 0.4^2)} = 0.828$$

where $\beta_{C,D}$ is given in Table D.6 in APPENDIX D. The values of β_{CAP}, $\beta_{T,C}$ and X have been discussed in Section 2.3.4.

The fragility curve for a one-*storey* SMF in the MH *seismic zone* is thus determined using the following equation and is shown in Figure 2.7:

$$P[Complete\ damage|S_d] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{S_d}{S_{d,m}}\right)\right] = \Phi\left[\frac{1}{0.828}\ln\left(\frac{S_d}{6.93}\right)\right]$$

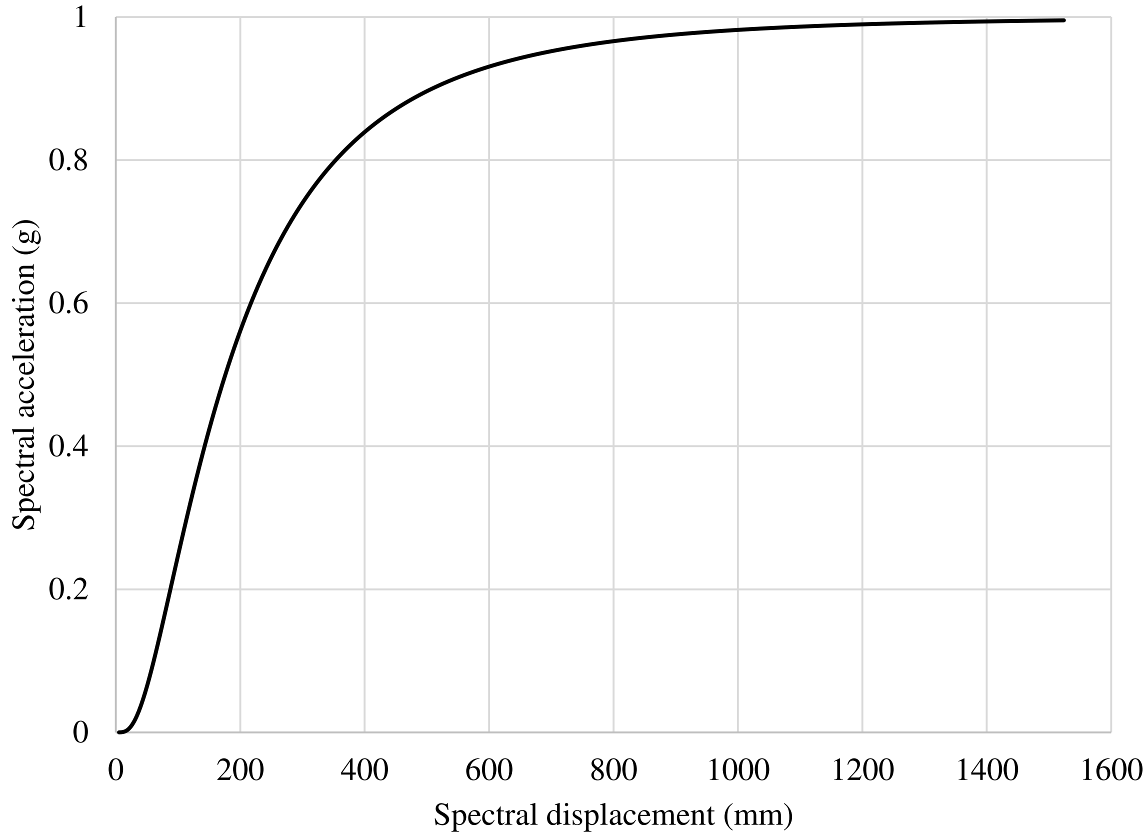

Figure 2.7: Fragility curve for a one-storey SMF in MH *seismic zone*

***Step 5*: Determine probability of collapse**

The probability of the structure being in the complete *damage* state is calculated by:

$$P[Complete\ damage|S_d = 0.784] = \Phi\left[\frac{1}{0.828}\ln\left(\frac{0.784}{6.93}\right)\right] = 0.0042$$

Therefore, the *building's* probability of collapse in the MH *seismic zone* is given by:

$$P(COL|CLE) = CF \times P[Complete\ damage|S_d = 0.784] = 0.08 \times 0.0042 = 0.000336$$

where $CF = 0.08$ is obtained from Table D.7 in APPENDIX D.

***Step 6*: Relate probability of collapse to structural score**

The structural score for a one-*storey* SMF in the MH *seismic zone* is determined by:

$$S = -\log_{10}\big(P(COL|CLE)\big) = -\log_{10}(0.000336) = 3.47$$

***Step 7*: Determine structural basic score**

Repeat Steps 1 through 6 for two-*storey* and three-*storey* SMF in the MH *seismic zone* to obtain two additional structural scores. The S_B score for SMF *model building type* in the MH *seismic zone* is taken as the average of the one- through three-*storey* score results, i.e.:

$$S_B = \frac{3.47 + 3.67 + 3.63}{3} = 3.6$$

2.4.5 Review of structural basic scores

Structural basic scores S_B are calculated for each of the sixteen *model building types* in all *seismic zones* following the procedure in Section 2.4.4 and are presented in Table 2.11. Generally speaking, wood *building types* have the best seismic performance in all *seismic zones*; concrete and masonry *building types* perform the worst in Moderately High and higher *seismicity*. The trend is consistent with common sense that wood and steel *buildings* (excluding SIW) experience less *damage* than concrete and masonry buildings in high *seismicity* due to their higher ductility capacity to dissipate inelastic energy, as well as their larger drift capacity.

Table 2.11: Structural basic scores for sixteen *model building types* in all *seismic zones*

MBT	Seismic zone						
	VL	L	M	MH	H	VH	VHX
WLF	10.4	8.4	6.7	5.3	4	2.8	2.4
WPB	10.4	8.4	6.6	5.2	3.8	2.6	2.2
SMF	7.3	5.6	4.4	3.6	2.7	2	1.8
SBF	7.7	5.9	4.5	3.8	2.8	2	1.7
SLF	8	6.2	4.8	3.9	3.1	2.2	1.9
SCW	8.3	6.5	4.9	4	2.9	2	1.7
SIW	8	6.2	4.6	3.3	2	1.4	1.3
CMF	6.7	5.1	3.8	2.9	2	1.4	1.2
CSW	8.5	6.6	4.9	3.9	2.7	1.8	1.5
CIW	7.5	5.8	4.1	3	1.7	1.2	1
PCW	8	6.2	4.6	3.8	2.5	1.6	1.3
PCF	7.4	5.7	4	3.1	2	1.3	1.1
RML	8.1	6.3	4.5	3.7	2.5	1.6	1.4
RMC	8.1	6.3	4.5	3.7	2.5	1.6	1.4
URM	7.4	5.7	3.9	2.8	1.5	1	0.9
MH	8.5	6.7	5.6	4.1	2.5	1.8	1.6

As *seismicity* increases, S_B scores drop, indicating increasing probabilities of *building* collapse. The increase is a result of the greater increasing rate of *building's* spectral displacement compared to the *building's* increased drift capacities. Nevertheless, in Very Low through High *seismic zones*, the S_B values are above or equal to acceptable structural threshold $S_{TH} = 2.0$ for *medium consequences of failure* (see Section 2.6) for sixteen *model building type*s except CIW and URM in High *seismic zone*. The inferior seismic performances of CIW and URM *buildings* have long been recognized. These *buildings* have been restricted to a maximum height of three *storeys* since the NBC 1975, and have been prohibited in moderate and higher *seismicity* areas since the NBC 2005. It is also noticed that, in High *seismic zones*, the S_B values for SIW, CMF, and PCW *buildings* are exactly equal to the acceptable structural threshold $S_{TH} = 2.0$. According to Mitchell *et al.* (1995), the seismic performance of SIW *buildings* is expected to be similar to CIW *buildings* (damage observed in high *seismicity* due to insufficient amounts of shear and confinement reinforcement in the columns), and similar to PCF *buildings* (damage observed in high *seismicity* due to inadequate connection of the diaphragm to the SFRS, loss of support, and spandrel beam failure). Therefore, it is reasonable that low S_B scores for SIW, CMF and PCF are the same as for CIW, and URM *buildings* in High *seismic zone*.

In the Very High *seismic zone*, S_B values for the majority of *model building type*s, including all concrete and masonry structures, are less than $S_{TH} = 2.0$. On one hand, seismic hazards are extremely high in this *seismic zone*, e.g., $S_a(0.2) = 1.40\ g$ and $S_a(1.0\ ) = 0.7\ g$ (Table 2.10). On the other hand, *building* capacity and fragility curves for *buildings* in Very High *seismic zone* are taken as the same as those for *buildings* in High *seismic zone*. Given the higher seismic demand and lack of specific *building* capacity and fragility curves in the Very High *seismic zone*, the S_B values are deemed reasonable.

2.5 Structural score modifiers

Eleven structural score modifiers are considered to address the following conditions affecting the seismic performance of *existing buildings*: (1) *building irregularity*, (2) *building* design period, (3) *original building importance*, (4) *Site Class*, (5) *building* height, (6) *building deterioration* and age, (7) *redundancy*, (8) *pounding*, (9) *seismic upgrading*, (10) *remaining occupancy time*, and (11) seismic hazard.

Table 2.12 presents the *building* conditions and relevant parameters in the structural scoring methodology. Given a specific *building* condition, the parameters with check marks in Table 2.12 indicate they would be affected by the *building* condition. In the following, the details of how all eleven score modifiers were determined are presented.

Table 2.12: *Building* conditions and key parameters in the score modifier calculations

Building conditions	Building parameters							
	Elastic response spectrum	**Building capacity**			**Demand spectrum**	**Seismic fragility**		**Building collapse**
		Yield capacity	Overstrength	Ductility	Degradation	Median drift capacity	Standard deviation	Collapse factor,
Building irregularity						✓		✓
Building design period		✓	✓	✓	✓	✓	✓	
Original building importance		✓		✓		✓	✓	
Site Class	✓							
Building height		✓	✓	✓		✓	✓	
Building deterioration and age		✓			✓	✓		✓
Redundancy						✓		✓
Pounding						✓		✓
Seismic upgrading		✓	✓	✓	✓	✓	✓	
Remaining occupancy time	✓							
Seismic hazard	✓							

2.5.1 Building irregularity

Building irregularities adversely affect the *building's* seismic performance by concentrating the demands at certain floor levels or elements, which can lead to building damage or collapse. *Building irregularities* are generally divided into two types: vertical *irregularities* and horizontal *irregularities*. The *building* irregularity modifier is determined for vertical *irregularities* (including moderate and severe) and horizontal *irregularities* by using the adjusted values of affected parameters in Table 2.12.

2.5.1.1 Vertical irregularity

Buildings with significant physical discontinuities in the distribution of mass, strength, stiffness, and geometry over their height are identified to have a vertical *irregularity* (FEMA 356 2000). In this section, vertical *irregularities* in FEMA P-154 are reviewed and then mapped to vertical *irregularities* in the NBC 2015.

In FEMA P-154, vertical *irregularities* are divided into *severe vertical irregularities* and *moderate vertical irregularities* based on their relative severity on a *building's* seismic performance. This division aims to address criticism that the vertical *irregularity* modifiers were overly severe (FEMA P-155 2015). All FEMA *building types* with the following characteristics are considered to have moderate vertical *irregularities* (except W1 *buildings* located on sloping sites):

- ***Sloping site:*** *Buildings* located on sloping sites, such as steep hills, have moderate vertical *irregularities* due to the presence of vertical setbacks that cause a stiffness discontinuity between the lower side and uphill side of the structure. For W1 *buildings*, the effect of sloping site is more severe, and it should be considered as a severe *irregularity*. For non-W1 *buildings*, there exists moderate sloping *irregularity* if there is at least a full *storey* grade change from one side of the *building* to the other.

- ***In-plane setback:*** Structures with an in-plane discontinuity of the SFRS, which can occur when the SFRS at a given level is offset relative to the SFRS at a lower level. This can in turn cause *damage* in the horizontal elements that connect the offset lateral elements and in the vertical elements that occur below the lateral elements at the upper levels (FEMA P-154 2015). This type of *irregularity* is generally not considered to exist in braced frames and moment-resisting frames due to connection design (NRC 2015).

- ***Split levels:*** Adjacent floor or roof levels in one part of a *building* that do not align with the levels on another part, otherwise known as split levels, create discontinuities in the diaphragm, which increases torsional deformation of the structure, thereby damaging the supporting elements (FEMA 310 1998).

All FEMA *building types* with one or more of the following features are said to have *severe vertical irregularities*:

- ***Weak and/or soft storey:*** A weak *storey* exists when one *storey* has less strength (fewer walls or columns) than the *storey* above or below it. A soft *storey* exists if the stiffness of one *storey* is dramatically less than that of most of the others.

- ***Out-of-plane setback:*** Out-of-plane setback is a discontinuity in the SFRS occurring in *buildings* with irregular vertical column spacing and misaligned vertical bracing in framing systems.

- ***Short column:*** Short columns, where the height/depth ratio is 50% of adjacent columns on a given level, attract greater seismic forces due to high relative stiffness and tend to suffer shear-related distresses (FEMA 310 1998).

In the NBC 2015, six types of vertical *irregularities* are considered and are summarized as follows:

1. ***Vertical stiffness irregularity:*** Vertical stiffness *irregularity* shall be considered to exist when the lateral stiffness of the SFRS in a *storey* is less than 70% of the stiffness of any adjacent *storey*, or less than 80% of the average stiffness of the three *storeys* above or below (equivalent to *soft storey* in FEMA P-154).

2. ***Weight (mass) irregularity:*** Weight *irregularity* shall be considered to exist where the weight, W_i, of any *storey* is more than 150% of the weight of an adjacent *storey*. A roof that is lighter than the floor below need not be considered (equivalent to weight [mass] *irregularity* in ASCE/SEI 7-16).

3. ***Vertical geometric irregularity:*** Vertical geometric *irregularity* shall be considered to exist where the horizontal dimension of the SFRS in any *storey* is more than 130% of that in an adjacent *storey* (included in *out-of-plane setback* in FEMA P-154).

4. ***In-plane discontinuity in vertical lateral-force-resisting element:*** Except for braced frames and moment-resisting frames, an in-plane discontinuity shall be considered to exist where there is an offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the *storey* below (equivalent to *in-plane setback* in FEMA P-154).

5. ***Out-of-plane offsets:*** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS (equivalent to *out-of-plane setback* in FEMA P-154).

6. ***Discontinuity in capacity – weak storey:*** A weak *storey* is one for which the *storey* shear strength is less than that for the *storey* above. The *storey* shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the *storey* shear for the direction under consideration (equivalent to *weak storey* in FEMA P-154).

While the weight (mass) *irregularity* is addressed in the NBC 2015, it is not considered in the structural scoring methodology given the challenges to quantitatively evaluate the masses of a *building's* storeys by exterior and interior view, without detailed calculations. Studies have been conducted to investigate the effect of mass *irregularity* on *building's* seismic response (Al-Ali and Krawinkler 1998; Tremblay and Poncet 2005; Magliulo et al. 2002). The results concluded that the negative effect of mass *irregularity* is limited. Other studies by Choi (2004) and Karavasilis et al. (2008), however, demonstrated that mass *irregularity* is an important factor affecting the *building's* seismic response.

In the NBC 2015, a new type of *irregularity* "gravity-induced lateral demand *irregularity*" was introduced. This *irregularity* shall be considered to exist where ratio α, calculated in accordance with Eq. (2.26), exceeds 0.1 for an SFRS with self-centring characteristics and 0.03 for other systems.

$$\alpha = Q_G/Q_y \tag{2.26}$$

where Q_G means gravity-induced lateral demand on the SFRS at the critical level of the yielding system, and Q_y means the resistance of the yielding mechanism required to resist the minimum earthquake loads, which need not be taken as less than R_o multiplied by the minimum lateral earthquake force as determined in Article 4.1.8.11 or 4.1.8.12 of Part 4 of Division B of the NBC 2015, as appropriate.

The determination of gravity-induced lateral demand *irregularity* involves structural analysis; therefore, it is impossible to be identified in a visual screening procedure used in the *Level 2 – SQST*. Similar to the weight (mass) *irregularity*, this *irregularity* is not considered in the structural scoring methodology.

It is noticed that the NBC 2015 does not explicitly address sloping site, split levels, and short column *irregularities*. In the 1993 NRC screening manual, sloping site is identified as vertical *irregularity* while short column is considered as another type of *irregularity*; short column *irregularity* is considered more severe than sloping site *irregularity*. When sloping site *irregularity* exists, the stiff short columns along the uphill attract more seismic demand, and thus impair the seismic performance of SFRS. When short column *irregularity* exists, the short columns attract more lateral load than they were designed for, which increases the potential for loss of vertical support and subsequent collapse. Split levels cause diaphragm discontinuity and thus tend to *damage* the supporting elements. In the structural scoring methodology, sloping site and split levels are treated as *moderate vertical irregularities* while short column is considered as a *severe vertical irregularity*, which is consistent with FEMA P-154.

In summary, five types of *vertical irregularities* (excluding weight [mass] *irregularity*) described in the NBC 2015, i.e., points 1, 3, 4, 5, and 6 above, as well as three additional *irregularities* discussed in FEMA P-154 (2015), namely *sloping site*, *split levels*, and *short column* are addressed. These eight *irregularity* types are divided into *moderate* and *severe vertical irregularities* and are summarized in Table 2.13.

Table 2.13: Vertical *irregularities* and their severity levels

Source	*Level 2 – SQST*	Severity
NBC 2015	vertical stiffness *irregularity*	severe
	vertical geometric *irregularity*	severe
	in-plane discontinuity	moderate
	out-of-plane offsets	severe
	discontinuity in capacity	severe
FEMA P-154	sloping site	moderate
	split levels	moderate
	short column	severe

The *vertical irregularity* modifiers are calculated separately for *moderate* and *severe vertical irregularities*. These are computed by modifying the fragility curve by decreasing *storey* drift ratio Δ_c, while increasing modal shape factor α_3 and collapse factor CF. The adjusted parameter values are tabulated in APPENDIX D.

If one or more *severe vertical irregularities* have been identified, the *severe vertical irregularity* modifier should be applied to the structural basic score. If one or more *moderate vertical irregularities* have been identified, and no *severe vertical irregularities* exist, the *moderate vertical irregularity* modifier should be applied to the structural basic score.

2.5.1.2 Horizontal irregularity

A horizontal *irregularity* is a condition that results in an irregular distribution of mass, strength, stiffness, and geometry over the *building's* plan dimension. In this section, horizontal *irregularities* in FEMA P-154 are reviewed and then mapped to horizontal *irregularities* in the NBC 2015.

In FEMA P-154, horizontal *irregularity* is considered if one or more of the following characteristics are identified:

- **Torsion:** This condition applies when a *building* has a definable or good lateral-load resistance in one direction but not the other, or when there are major stiffness eccentricities in the SFRS that may cause twisting (torsion) around a vertical axis.
- **Non-parallel systems:** Wedge-shaped buildings, triangular in plan and located on corners of streets not meeting at 90 degrees, are similarly susceptible to torsion and increased damage and collapse potential.
- **Re-entrant corner:** Irregularly shaped *buildings*, such as L, C, E, T and **+**, are likely to suffer local diaphragm *damage* due to stress concentrations at re-entrant corners (FEMA P-

154 2015). As a general rule, the re-entrant corners are considered as horizontal *irregularities* when these corners project more than 20 feet (6 meters).

- **Large diaphragm opening:** Large openings in the floors or roof can compromise the ability of the diaphragms to transfer seismic forces. As a general rule, floors and roofs with large openings, typically greater than 50% of the plan dimension, are considered to be horizontal *irregularities* (FEMA P-154 2015).

- **Beams not aligned with columns:** In some buildings, typically concrete buildings, beams do not align with the columns in plan. It is considered to be a horizontal *irregularity* if observed from structural drawings or field inspection.

In the NBC 2015, two types of horizontal *irregularities* are considered and are summarized as follows:

1. **Torsional sensitivity** (to be considered when diaphragms are not flexible): Torsional sensitivity shall be considered to exist when ratio B calculated according to Sentence 4.1.8.11.(10) exceeds 1.7.

2. **Non-orthogonal systems:** A non-orthogonal system *irregularity* shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes (equivalent to non-parallel *irregularity* in FEMA P-154).

It is acknowledged that some symmetric non-orthogonal systems (e.g., octagonal and round structures) may be more resistant to torsional effects and behave better than regular *buildings*. However, it is difficult to assess this in a visual screening procedure. Considering the conservative nature of the structural scoring methodology, it is reasonable to penalize all non-orthogonal systems in the same manner.

It is noticed that the NBC 2015 does not explicitly address the horizontal *irregularities* caused by re-entrant corners, large diaphragm openings, and beams not aligned with columns. Given the detrimental effects of these *building* configurations on a *building's* seismic performance, these three types of *irregularities* are considered in the structural scoring methodology, which is consistent with FEMA P-154.

Compared to two separate vertical *irregularity* modifiers (i.e., moderate and severe), a single horizontal *irregularity* modifier is calculated for each of the sixteen *model building types,* in each *seismic zone,* by reducing the *storey* drift ratio Δ_c of a typical *building* by 50% in order to reflect the decreased ductility capacity and increase collapse factor CF by a factor of 2, to reflect an increase in the portions of the *buildings* with horizontal *irregularities* that are likely to collapse. The adjusted parameter values are provided in Table D.4 and Table D.7 in APPENDIX D.

If one or more horizontal *irregularities* have been identified, a horizontal *irregularity* modifier is applied. Manufactured homes (MH) do not usually contain horizontal *irregularities* (FEMA P-154 2015); therefore, the horizontal *irregularity* modifier does not apply to these *buildings*.

2.5.2 **Building design period**

General trends observed from previous major earthquakes showed that *buildings* designed according to newer *building* code editions perform better than those designed according to older editions. There are two key NBC editions identified in the *Level 2 – SQST*, i.e., *pre-code NBC edition* and *benchmark NBC edition*. Based on these two key NBC editions, *existing buildings* are grouped into three *building design periods*: *pre-code*, *pre-benchmark*, and *post-benchmark*. The *building* design period modifier intends to assess the effects of *building design periods* on the seismic performance of *existing buildings*.

The *pre-code NBC edition* and *benchmark NBC edition* for each of the sixteen *model building types* are provided in Table 2.3. Two score modifiers, i.e., *pre-code* modifier, and post-benchmark modifier, are calculated based on *pre-code* parameter values and post-benchmark parameter values in APPENDIX D, respectively. The *pre-code* modifier is applied to the structural basic score if the *building* was designed and constructed prior to the *pre-code NBC edition*. The post-benchmark modifier is applied to the structural basic score if the *building* was designed and constructed to an applicable *benchmark NBC edition* or after.

The *pre-code* modifier does not apply to the *buildings* in Very Low and Low *seismicity* because structural basic scores of *buildings* located in these two *seisimic zones* are determined based on *pre-code* parameter values. The post-benchmark modifier does not apply to SIW (steel frame with infill masonry walls), CIW (concrete frame with infill masonry walls) or URM (unreinforced masonry bearing walls) because no *benchmark NBC editions* are specified for these *buildings*.

2.5.3 **Original building importance**

The *importance category* can influence the seismic performance of *buildings* given that importance factors corresponding to different importance categories are incorporated into the design base shear and that restrictions and limitations are imposed in the seismic design. For example, *buildings* essential to the provision of services in the event of a disaster, such as hospitals, power generating stations and water treatment facilities are constructed in accordance with a higher standard of seismic safety than ordinary *buildings*, such as office *buildings* and retail facilities (HAZUS TM 2003). The *original building importance* modifier aims to assess the effect of the *original building importance* category on the seismic performance of *existing buildings*. In this section, existing approaches to address the effect of *original building importance* are reviewed first. The HAZUS TM approach is then adopted with modifications to determine the *original building importance* modifiers for different importance categories in the NBC 2015.

In FEMA P-154 (2015), seismic screening of *high importance buildings* was considered, with the focus on *building* collapse. Where a higher seismic performance objective such as functionality is required, detailed seismic evaluation is triggered (FEMA P-155 2015). It should be emphasized that FEMA P-154 treats *high importance buildings* as ordinary *buildings* (equivalent to *Normal importance buildings* in Canada) in the seismic screening, disregarding the effect of *building* importance on a *building's* seismic performance. In the 1993 NRC screening manual and its subsequent revision by Saatcioglu et al. (2013), a *building* importance factor *E* is established to account for the consequences of collapse or failure of critical or high-*occupancy buildings*. *Building* importance is not an explicit consideration in the method by Tischer et al. (2014), although it is implicitly reflected as their methodology was developed for the evaluation of school *buildings* (i.e., *High importance buildings*). Conversely, the screening method proposed by Karbassi and Nicollet (2008) focuses on *Normal importance buildings* and does not provide a process for screening structures in other importance categories. In HAZUS TM (FEMA 2012b), an importance factor of I=1.5 is explicitly considered in calculating the probabilities of collapse of Special *buildings* (equivalent to *Post-disaster buildings* in Canada). Note that HAZUS TM provided an approach to modify the *building* capacity and fragility curves for each *model building type* and each *seismic zone*. Therefore, the HAZUS TM methodology is adopted with modifications to calculate the *original building importance* modifier in the structural scoring methodology. In the following, a brief introduction of the HAZUS TM methodology is presented.

2.5.3.1 HAZUS TM methodology

In the HAZUS TM methodology, the capacity curves for Special *buildings*, such as hospitals and emergency operations centres, are constructed by increasing the seismic design coefficient C_s by importance factor $I = 1.5$, and increasing ductility factor μ by 1.2 for Low-code *buildings* and by 1.33 for Moderate-code *buildings*, while keeping Pre-code *buildings* unchanged (Table 5.6 in HAZUS TM 2012b). The increase of C_s is equivalent to the increase of lateral force by applying importance factor, $I = 1.5$, because C_s is proportional with lateral force for specific *buildings*. The ductility increase results from the higher construction quality requirement for Special *buildings*, as shown in Table 2.14 (Table 2.3 in HAZUS AEBM 2012a). For example, in ordinary construction (with respect to ordinary *buildings*), a *building* is designed at Moderate-code design level, however, in superior construction (with respect to Special *buildings*), this *building* is similar to High-code design level. Therefore, ductility factors are multiplied by 1.33 to account for the higher construction quality requirement. Since ductility factors for Special High-code level are unavailable in Table 5.6 of HAZUS TM, they are conservatively taken as the same as those for High-code design level. For the sake of brevity, the reader is directed to Section 2.2.4 for a detailed discussion on design code levels.

Table 2.14: Approximate relationship between design code level and construction quality

Construction quality	Design code level			
	High-code	Moderate-code	Low-code	None
Superior	Special High-code	High-code	Moderate-code	Low-code
Ordinary	High-code	Moderate-code	Low-code	Pre-code
Inferior	Moderate-code	Low-code	Pre-code	Pre-code

In *building* codes such as the IBC 2018, allowable interstorey drift limits for Risk Category IV ($I = 1.5$) are less than those for Risk Category II *buildings* ($I = 1.0$), because a more stringent drift capacity requirement is applied. Nevertheless, the actual interstorey drift capacities of Special *buildings* (equivalent to Risk Category IV *buildings*) are assumed to be a slightly higher than those of ordinary *buildings* (equivalent to Risk Category II *buildings*) (page 6-12 in HAZUS TM 2012b). It is difficult to quantify this improvement in interstorey drift capacity since it is a function not just of *building type* and design parameters such as ductility increase, but also of design review and construction inspection. HAZUS TM assumes that the improvement in displacement capacity results in increase in drift capacity by a factor of 1.25 for all Special *buildings* in all design code levels. In addition, a reduction in *building* capacity variability is recognized (page 6-13 in HAZUS TM 2012b).

It is noted that HAZUS TM considered only special *buildings*. In addition, the logarithmic standard deviations for developing fragility curves are dependent on design code level, which is not consistent with the structural scoring methodology where the logarithmic standard deviations are independent of design code level. Therefore, a few modifications are made in the structural scoring methodology to resolve the inconsistency and determine the *original building importance* modifiers for all importance categories in the NBC 2015.

2.5.3.2 Determination of original building importance modifier

Determination of *original building importance* modifier for *post-disaster buildings*

The HAZUS TM methodology is adopted to determine the *original building importance* modifier for *Post-disaster buildings*, M_{PD}. Since Canadian Post-disaster *buildings* are equivalent to Special *buildings* in the U.S., the *Post-disaster* importance category corresponds to the superior construction quality in Table 2.14. Table 2.15 provides a mapping between *seismic zones* and design code levels for different construction quality. (Note that ordinary construction corresponds to *Normal importance* category.)

Table 2.15: Mapping between *seismic zones* and design code levels for different construction qualities

Seismic zone	**Construction quality**		
	Inferior	Ordinary	Superior
Very Low	Pre-code	Pre-code	Low-code
Low	Pre-code	Pre-code	Low-code
Moderate	Pre-code	Low-code	Moderate-code
Moderately High	Average of Pre- and Low-code	Average of Low- and Moderate-code	Average of Moderate- and High-code
High	Low-code	Moderate-code	High-code
Very High	Low-code	Moderate-code	High-code

As HAZUS TM did, an importance factor $I_E = 1.5$ is applied to C_s and values 1.33, 1.2, and 1.0 are applied to ductility factors, μ, for Moderate-code, Low-code, and Pre-code design levels to develop *building* capacity curves. The μ for various design code levels are taken as those for calculating structural basic scores in Table D.1in APPENDIX D. The values 1.33 and 1.2 reflect the superior quality of construction for *Post-disaster buildings*. It is noted that μ for Pre-code and Low-code design levels are the same. According to the mapping in Table 2.15, the average values of 1.2 and 1.33 are applied to the ductility factors for *buildings* in the Moderately High *seismic zone*.

In developing fragility curves that correspond to the structural complete *damage* state, the median spectral displacements are taken as 1.25 times those for *Normal importance buildings*. $\beta_{C,D}$ is decreased due to the reduction of variability in capacity variability ($\beta_{CAP} = 0.15$ for Special *buildings* compared to $\beta_{CAP} = 0.25$ for ordinary *buildings*, according to HAZUS TM 2012b). By taking $\beta_{CAP} = 0.15$ and $\beta_{T,C} = 0.4$, a "generic" standard deviation $\beta_{C,D} = 0.9$ is determined from HAZUS AEBM (2012a). In the structural scoring methodology, a generic value $\beta_{C,D} = 0.9$ is adopted in developing fragility curves of low-rise *Post-disaster buildings*. For mid- and high-rise *post-disaster buildings*, $\beta_{C,D} = 0.9$ decreases by 0.01 with the increase in the number of *storeys,* until it approaches its lower bound value of $\beta_{C,D} = 0.8$. The method to determine $\beta_{C,D}$ as a function of the number of *storeys* is adopted from FEMA P-154 (2015) and OSHPD HAZUS (2016).

Investigation of scale factors for *building* ductility and drift capacities

The approach for determining the *original building importance* modifier for *High importance* and *Low importance buildings* is not provided in HAZUS TM. It is noticed that, for Special *buildings* in HAZUS TM, scale factors used for increasing ductility and drift capacities are not equal to *building* importance factor $I_E = 1.5$. The influences of these two factors on the *original building importance* modifier for *Post-disaster buildings* are investigated to acquire valuable insight in accounting for *High* and *Low importance buildings*. Three cases (namely Base Case, Case 1, and Case 2) are considered for achieving this goal. The corresponding scale factors for C_s and μ are summarized in Table 2.16.

Table 2.16: Scale factors for seismic coefficient, ductility, and median spectral displacement

Case	Seismic coefficient (C_S)	Ductility factor μ				Median spectral displacement (S_d)
		VL, L	M	MH	H, VH, VHX	
Base case	$1.5 \times C_S$	$1.0 \times \mu$	$1.2 \times \mu$	$1.27 \times \mu$	$1.33 \times \mu$	$1.25 \times S_d$
Case 1	$1.5 \times C_S$	$1.33 \times \mu$				$1.25 \times S_d$
Case 2	$1.5 \times C_S$	$1.0 \times \mu$	$1.2 \times \mu$	$1.27 \times \mu$	$1.33 \times \mu$	$1.5 \times S_d$

The *original building importance modifiers* for *Post-disaster buildings* for each of the sixteen *model building types* in each *seismic zone* are calculated and tabulated in Table 2.17. The results show that the scale factor for ductility has a negligible effect on the *original building importance modifier*. Therefore, for simplicity, the ductility factors for *post-benchmark buildings* (Table D.1) are used to calculate the *original building importance modifier*. The scale factor for the median spectral displacement, however, has a significant influence on the *original building importance modifier*. If a scale factor of 1.5 ($I_E = 1.5$) is used to scale the drift capacity, non-conservative values of the *original building importance modifier* would be obtained; hence, this scale factor should be taken with caution corresponding to importance categories other than the *Post-disaster* category.

Table 2.17: ***Original building importance*** **modifiers for** ***Post-disaster buildings*** **corresponding to three cases**

Seismicity	Cases	WLF	WPB	SMF	SBF	SLF	SCW	SIW	CMF	CSW	CIW	PCW	PCF	RML	RMC	URM	MH
VL	Base case	1.5	1.5	1.1	1.1	1.2	1.2	1.2	1.0	1.2	1.1	1.2	1.1	1.2	1.2	1.1	1.2
	Case 1	1.5	1.5	1.1	1.1	1.2	1.2	1.2	1.0	1.2	1.1	1.2	1.1	1.2	1.2	1.1	1.2
	Case 2	2.1	2.1	1.6	1.7	1.7	1.8	1.7	1.5	1.8	1.6	1.8	1.6	1.8	1.8	1.6	1.8
L	Base case	1.2	1.2	0.8	0.9	0.9	0.9	0.9	0.8	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
	Case 1	1.2	1.2	0.8	0.9	0.9	0.9	0.9	0.8	1.0	0.9	0.9	0.9	1.0	1.0	0.9	0.9
	Case 2	1.7	1.7	1.3	1.3	1.4	1.4	1.4	1.2	1.5	1.3	1.4	1.3	1.4	1.4	1.3	1.4
M	Base case	1.1	1.2	0.7	0.8	0.8	1.0	1.0	0.7	1.2	1.1	1.0	1.1	1.2	1.2	1.1	1.2
	Case 1	1.1	1.2	0.7	0.8	0.8	1.0	1.0	0.7	1.2	1.1	1.0	1.1	1.2	1.2	1.1	1.2
	Case 2	1.7	1.7	1.1	1.3	1.2	1.5	1.4	1.1	1.7	1.5	1.4	1.5	1.7	1.7	1.6	1.6
MH	Base case	0.9	1.0	0.6	0.6	0.6	0.7	0.8	0.5	0.9	0.7	0.7	0.7	0.9	0.9	0.8	0.9
	Case 1	0.9	1.0	0.6	0.6	0.6	0.7	0.8	0.5	0.9	0.7	0.7	0.7	0.9	0.9	0.8	0.9
	Case 2	1.4	1.4	0.9	1.0	1.0	1.1	1.1	0.8	1.3	1.1	1.1	1.1	1.2	1.2	1.1	1.2
H	Base case	0.8	1.0	0.5	0.6	0.5	0.8	0.5	0.5	0.9	0.5	0.9	0.7	0.9	0.9	0.5	0.6
	Case 1	0.8	1.0	0.5	0.6	0.5	0.8	0.5	0.5	0.8	0.5	0.9	0.7	0.8	0.8	0.5	0.6
	Case 2	1.2	1.4	0.8	0.9	0.8	1.2	0.7	0.7	1.2	0.7	1.2	0.9	1.2	1.2	0.6	0.8
VH	Base case	0.8	0.7	0.4	0.5	0.6	0.5	0.3	0.3	0.5	0.3	0.5	0.4	0.5	0.5	0.2	0.4
	Case 1	0.7	0.7	0.4	0.4	0.6	0.5	0.3	0.3	0.5	0.3	0.5	0.3	0.5	0.5	0.2	0.4
	Case 2	1.1	1.0	0.6	0.7	0.8	0.7	0.4	0.5	0.8	0.4	0.7	0.5	0.7	0.7	0.3	0.6
VHX	Base case	0.7	0.6	0.3	0.4	0.5	0.4	0.2	0.2	0.4	0.2	0.4	0.2	0.3	0.3	0.1	0.3
	Case 1	0.7	0.6	0.3	0.4	0.5	0.4	0.2	0.2	0.4	0.2	0.4	0.2	0.3	0.3	0.1	0.3
	Case 2	0.9	0.8	0.5	0.5	0.7	0.5	0.3	0.3	0.6	0.2	0.5	0.4	0.5	0.5	0.2	0.4

Determination of *original building importance* modifier for *High importance buildings*

Considering that the *High importance* category is between *Normal importance* and *Post-disaster* categories, it is reasonable to deduce that, for *High importance buildings* ($I_E = 1.3$), the scale factor for ductility, μ,, should be between 1.0 (i.e., *Normal importance*) and 1.33 (i.e., *Post-disaster*); the scale factor for the median spectral displacement, S_d, should be between 1 (i.e., *Normal importance*) and 1.25 (i.e., *Post-disaster*); the logarithmic standard deviations, $\beta_{C,D}$, should be between values for *Normal importance buildings* and for *Post-disaster buildings*. Therefore, *original building importance* modifier values for *High importance buildings* are expected to be between 0 for *Normal importance buildings* and *Post-disaster buildings* values. Nevertheless, specific parameter values for *High importance category* are not provided in HAZUS TM or other relevant studies.

It has been observed that ductility has a negligible effect on the *original building importance* modifier; hence, for simplicity, ductility factors for *post-benchmark buildings* are used to calculate the *original building importance* modifier for *High importance buildings*. It is also found that, for *Post-disaster buildings*, the scale factor for median spectral displacement S_d is the average of $I_E = 1.0$ and $I_E = 1.5$; hence, it is reasonable to assume the scale factor for the S_d of *High importance buildings* to be 1.15 (i.e., the average of $I_E = 1.0$ and $I_E = 1.3$). In the following, two cases are investigated: 1) the *original building importance* modifier for *High importance buildings* is simply taken as the average of 0 for *Normal importance buildings* and the *original building importance* modifier for *Post-disaster buildings*, i.e., one half of the *original building importance* modifier for *Post-disaster buildings*; 2) the *original building importance* modifier for *High importance buildings* is calculated based on 1.3 C_s ($I_E = 1.3$), ductility factors for *post-benchmark buildings*, 1.15 S_d, and the average of $\beta_{C,D}$ values for *Normal importance buildings* and for *Post-disaster buildings*.

The results of these two cases are summarized in Table 2.18. In Very Low and Low *seismic zones*, both cases provide similar values; however, for higher *seismic zones*, Case 2 tends to provide larger values than Case 1. Nevertheless, for all *model building type*s in all *seismic zones*, the absolute differences of score modifiers from two Cases are less than or equal to 0.2. Therefore, for the purpose of simplicity and conservatism, Case 1 is used to determine the *original building importance* modifier for *High importance buildings*.

Table 2.18: *Original building importance* modifier for *High importance buildings* corresponding to two cases

Seismicity	Cases	WLF	WPB	SMF	SBF	SLF	SCW	SIW	CMF	CSW	CIW	PCW	PCF	RML	RMC	URM	MH
VL	**Case 1**	0.8	0.8	0.6	0.6	0.6	0.6	0.6	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
	Case 2	0.8	0.8	0.6	0.6	0.6	0.7	0.6	0.6	0.7	0.6	0.7	0.6	0.7	0.7	0.6	0.6
L	**Case 1**	0.6	0.6	0.4	0.5	0.5	0.5	0.5	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
	Case 2	0.6	0.7	0.5	0.5	0.5	0.5	0.5	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
M	**Case 1**	0.6	0.6	0.4	0.4	0.4	0.5	0.5	0.4	0.6	0.6	0.5	0.6	0.6	0.6	0.6	0.6
	Case 2	0.6	0.7	0.4	0.5	0.4	0.6	0.6	0.4	0.8	0.7	0.6	0.8	0.8	0.8	0.8	0.7
MH	**Case 1**	0.5	0.5	0.3	0.3	0.3	0.4	0.4	0.3	0.5	0.4	0.4	0.4	0.5	0.5	0.4	0.5
	Case 2	0.5	0.6	0.3	0.3	0.4	0.4	0.5	0.3	0.5	0.5	0.4	0.5	0.6	0.6	0.6	0.6
H	**Case 1**	0.4	0.5	0.3	0.3	0.3	0.4	0.3	0.3	0.4	0.3	0.5	0.4	0.4	0.4	0.3	0.3
	Case 2	0.5	0.7	0.3	0.4	0.3	0.6	0.4	0.3	0.6	0.3	0.7	0.5	0.6	0.6	0.4	0.4
VH	**Case 1**	0.4	0.4	0.2	0.2	0.3	0.3	0.2	0.2	0.3	0.2	0.3	0.2	0.3	0.3	0.1	0.2
	Case 2	0.6	0.5	0.3	0.3	0.4	0.4	0.2	0.2	0.4	0.2	0.4	0.3	0.4	0.4	0.2	0.3
VHX	**Case 1**	0.4	0.3	0.2	0.2	0.3	0.2	0.1	0.1	0.2	0.1	0.2	0.1	0.2	0.2	0.1	0.2
	Case 2	0.4	0.4	0.2	0.3	0.4	0.3	0.2	0.2	0.3	0.1	0.3	0.2	0.3	0.3	0.1	0.2

Determination of *original building importance* modifier for *Low importance buildings*

The *Low importance* category corresponds to the inferior construction quality in Table 2.14. For *Low importance buildings* ($I_E = 0.8$), *building* capacity curves are developed based on $0.8C_s$ and ductility factors for *pre-code buildings*. The scale factor for the median spectral displacement is taken as 0.9 (the average of $I_E = 0.8$ and $I_E = 1.0$), accounting for the reduction of drift capacity due to inferior construction quality. Values of $\beta_{C,D}$ are taken as those values for *pre-code buildings*. The *original building importance* modifier values for *Low importance* buildings are calculated for each of the sixteen *model building types* in each *seismic zone*. Note that $\beta_{CAP} = 0.4$ and $\beta_{T,C} = 0.4$ are used to calculate β_d in Eq. (2.18).

2.5.3.3 Modified building capacity curves and median spectral displacements

Building capacity curves and median spectral displacements with respect to the structural complete *damage state*, constructed for a one-*storey* SMF with respect to *Low*, *Normal*, and *Post-disaster* importance categories in the Moderately High *seismic zone*, are calculated and shown in Figure 2.8. *Building* capacity curves are not provided for the *High importance* category because the *original building importance* modifier for *High importance buildings* is not calculated based on modifying *building* capacity and fragility curves. The figure highlights the expectation that *buildings* with elevated importance category will exhibit increased strength and stiffness, and increased drift capacity with respect to the structural complete *damage* state.

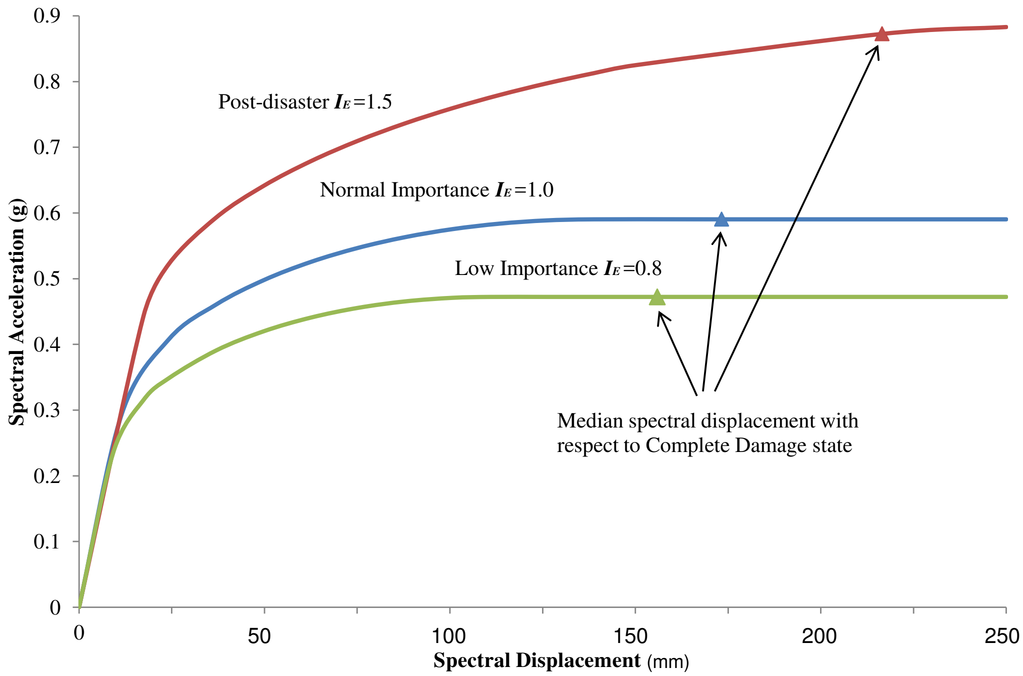

Figure 2.8: Modified *building* capacity curves and median spectral displacements with respect to structural complete *damage* state

2.5.4 Site Class

The soil properties at the *building* site are important factors that influence seismic vulnerability. Soft soils can significantly amplify long-period spectral response accelerations. For example, during the 1985 Mexico City earthquake, the maximum rock acceleration was amplified four times by a soft clay deposit that would have been classified as S_4 (category 4 in the NBC 1990), whereas the spectral amplitudes were about 15 to 20 times larger than on rock at a period near 2 seconds (FEMA 303 1997). In this section, the evolution of site classification is presented first and *Site Class* modifiers for different *Site Classes* are then determined.

The effect of soil properties on structural response was first addressed in the NBC 1965 by applying a foundation factor F to the design base shear. A value of 1.5 was assigned for *buildings* founded on highly compressible soil and a value of 1.0 was assigned for other soil conditions. The soil profiles were expanded to three in NBC 1975 and further expanded to four in the NBC 1990. Table 2.19 provides definitions of soil types and values of *F* as per the NBC 1995, which are the same as those in the NBC 1990. It is noteworthy that in the NBC 1995 and its previous editions, *F* solely depended on soil type, without considering the influence of ground shaking intensity and *buildings'* fundamental period on the structural response. It is also noted that the foundation factor *F* was not addressed in the seismic zoning defined based on peak ground acceleration or peak ground velocity. In this case, given a specified *seismic zone*, *buildings* within a specified soil category were designed with the same seismic design/detailing requirements. When subjected to earthquake motions, long-period *buildings* located on soft soils tend to drift more and are thus more prone to experiencing severe *damage*.

Table 2.19: Foundation factors (as per the NBC 1995)

Categories	Type and depth of rock and soil	*F*
1	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils, compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	1.0
2	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
3	Very loose and loose coarse-grained soils with depth greater than 15 m	1.5
4	Very soft and soft fine-grained soils with depth greater than 15 m	2.0

In the NBC 2005, a new site classification system proposed by the 1994 National Earthquake Hazard Reduction Program (NEHRP) Recommended Seismic Provisions (FEMA 1994) was adopted to describe the site conditions. Five *Site Classes*, A, B, C, D and E, were defined based on average site properties (i.e., shear wave velocity, average penetration resistance, and soil undrained shear strength) in the top 30 m. An extra *Site Class F* was defined as other soils such as liquefiable soils, as well as quick and highly sensitive clays. Compared to the foundation factor *F*, the new site coefficient *F(T)* is a function of *Site Class* and *buildings'* fundamental period. In the NBC 2015, an extra parameter, i.e. reference peak ground acceleration PGA_{ref}, was introduced to determine the site coefficient *F(T)*. PGA_{ref} is equal to 0.8 PGA, where the ratio $S_a(0.2)/\text{PGA} < 2.0$, and is equal to PGA otherwise. The new feature addressed the differences of empirical relationships (e.g., stress drop, regional attenuation, fault sizes, etc.) between Eastern North America and Western Canada and California (Motazedian 2018). It is also noted that $F(0.2)$ and $F(1.0)$ are incorporated into the thresholds for seismic design/detailing requirements (Table 4.1.8.9 in Part 4 of Division B of the NBC 2015). Table 2.20 presents $F(0.2)$ for *Site Classes* corresponding to various PGA_{ref} values. It is noticed that soft soil identified as *Site Class* E amplifies $S_a(0.2)$ in low to moderate *seismicity* while it reduces $S_a(0.2)$ in high *seismicity*.

Table 2.20: Site coefficients corresponding to 0.2-second period (Table 4.1.8.4.-B in the NBC 2015)

Site Class	Values of $F(0.2)$				
	$\mathrm{PGA_{ref}}\leq 0.1$	$\mathrm{PGA_{ref}}=0.2$	$\mathrm{PGA_{ref}}=0.3$	$\mathrm{PGA_{ref}}=0.4$	$\mathrm{PGA_{ref}}\geq 0.5$
A	0.69	0.69	0.69	0.69	0.69
B	0.77	0.77	0.77	0.77	0.77
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.09	1.00	0.94	0.90
E	1.64	1.24	1.05	0.93	0.85

FEMA P-154 (2015) determined the Soil Type (i.e. *Site Class* in Canada) modifier for a specified Soil Type using median spectral response accelerations, while keeping the *building* capacity curve unchanged. The *building* capacity curve represents the expected (i.e., mean) *building* capacity of a *building* inventory for a specific mode *building type* and *seismicity* region. This *building* inventory consists of *buildings* located in different Soil Types; therefore, the Soil Type effect has been incorporated into the development of the *building* capacity curve. In addition, the variability of *building* capacity including the Soil Type variability is considered in developing the fragility curve.

Similar to FEMA P-154, the structural scoring methodology calculates *Site Class* modifiers for different *Site Classes* using mean values of short- and long-period spectral accelerations provided in Table 2.21 while keeping the values of other parameters unchanged (Table 2.12). These mean values are calculated based on 679 locations in Appendix C of Division B of the NBC 2015. The procedure to determine the mean values of short- and long-period spectral accelerations corresponding to *Site Class* A are presented herein as an example:

1) Determine $\mathrm{PGA_{ref}}$ value for a given location in a Very Low *seismic zone*;

2) Determine site coefficients $F(0.2)$, $F(0.5)$ and $F(1.0)$ corresponding to the $\mathrm{PGA_{ref}}$ value;

3) Calculate short-period spectral acceleration $\mathrm{S}(0.2)$, whichever is greater of $F(0.2)S_a(0.2)$ and $F(0.5)S_a(0.5)$, and long-period spectral acceleration $S(1.0)$, i.e. $F(1.0)S_a(1.0\ )$;

4) Repeat steps (1) through (3) for other locations in a Very Low *seismic zone* to determine $\mathrm{S}(0.2)$ and $S(1.0)$;

5) Calculate mean values of $\mathrm{S}(0.2)$ and $S(1.0)$ in a Very Low *seismic zone*; and

6) Repeat step (5) to obtain the mean values of $\mathrm{S}(0.2)$ and $S(1.0)$ for all *seismic zones*.

Table 2.21: Mean values of short- and long-period spectral accelerations corresponding to five *Site Classes* in all *seismic zones*

Seismic zone	Site Class A		Site Class B		Site Class C (Basis of basic scores)		Site Class D		Site Class E	
	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$	$S(0.2)$	$S(1.0)$
Very Low	0.05	0.02	0.05	0.02	0.07	0.03	0.09	0.05	0.12	0.09
Low	0.10	0.04	0.11	0.04	0.14	0.07	0.17	0.10	0.24	0.18
Moderate	0.18	0.07	0.20	0.08	0.26	0.12	0.31	0.17	0.41	0.26
Moderately High	0.35	0.11	0.39	0.12	0.51	0.20	0.53	0.26	0.60	0.37
High	0.60	0.23	0.67	0.26	0.87	0.41	0.88	0.52	0.97	0.67
Very High	0.97	0.40	1.08	0.44	1.40	0.70	1.36	0.85	1.41	0.98
Very High Max	1.19	0.51	1.33	0.57	1.73	0.90	1.56	1.09	1.61	1.25

Similar to FEMA P-154, separate *Site Class* modifier values are calculated for *Site Class* E reflecting the fact that taller *buildings* on soft soil experience an apparent amplification of earthquake forces due to the resonance between ground vibrations and fundamental periods of these *buildings*, as was observed during the 1985 Mexico City earthquake (Waas 1991). *Buildings* located on *Site Class F*, such as liquefiable soils or quick and highly sensitive clays with the potential to severely affect the seismic performance of *existing buildings,* are flagged for *Level 3 – SEG*.

2.5.5 **Building height**

The *building* height modifier assesses the influence of *building* height on a *building's* seismic performance. *Buildings* of different heights are grouped into three categories, i.e. low-rise (1-3 storeys), mid-rise (4-7 storeys), and high-rise (>7 storeys), which is consistent with FEMA P-154. In this section, existing seismic screening methods for addressing the *building* height effect are introduced first. The methodology to determine the *building* height modifier in the structural scoring methodology is then presented.

The consideration of the *building* height effect varies between seismic screening methods. The 1993 NRC screening manual does not consider *building* height. In an update to the NRC screening manual, Saatcioglu et al. (2013) accounted for *building* height by establishing the period *T* used to calculate the *seismicity* factor *A*. Karabassi and Nollet (2008) developed score modifiers for mid- and high-rise *buildings* by modifying the capacity and fragility curves for different *building* heights following HAZUS AEBM (FEMA 2012a). Tischer et al. (2014) did not address *building* height since their screening methodology was developed for schools, which tend to be less than three *storeys* tall. In FEMA 154 (2002), two separate *building* height modifiers were determined for mid-rise (4~7 *storeys*) and high-rise (>7 *storeys*) *buildings*. The *building* capacity and fragility parameter values for these two *building* height configurations were adopted from HAZUS AEBM (2003a). *Building* height modifiers are not provided in FEMA P-154 (2015) because it is observed, in recent earthquakes, that low-rise *buildings* behave better than taller *buildings* (FEMA P-154

2015). Nevertheless, *building* height is considered in determining the Soil Type modifier for *Site Class* E to address the soil-structure interaction effect.

Based on the review of the existing methods, the *Level 2 – SQST* project team decided to adopt FEMA 154 (2002) to determine the *building* height modifier. Similar to FEMA 154, separate modifiers are calculated for mid-rise and high-rise *buildings*, respectively. *Building* properties needed in the calculations are provided in APPENDIX D. The procedure to calculate the *building* height modifier for mid-rise *buildings* is presented herein as an example:

1) Determine the structural scores for a specified *model building type* and *seismic zone* using *building* capacity and fragility parameter values for four-, five-, six-, and seven-storey *buildings*, respectively;

2) Determine the average structural score based on the structural scores determined in step (1);

3) Subtract the average structural score by the structural basic score to obtain the *building* height modifier for mid-rise *buildings* corresponding to the specified *model building type* and *seismic zone*; and

4) Repeat steps 1 through 3 to determine the *building* height modifier for all *model building types* and *seismic zones*.

It is noted that, in the NBC 2015, the number of *storeys* in wood *buildings* is limited to six. In FEMA P-154, however, wood *building*s are limited to five *storeys*. Since FEMA P-154 did not provide guidance on how to calculate the fundamental periods for wood *buildings*, a sensitivity analysis is performed to investigate how the variations in *buildings'* fundamental periods influence the structural score. Given the periods of one- to five-*storey* wood *buildings*, namely 0.35 s, 0.38 s, 0.49 s, 0.60 s and 0.70 s (see Table D.1 in APPENDIX D), the period of six-*storey* wood *buildings* (including WLF and WPB) is considered to be between 0.75 s and 0.85 s by judgement. The structural scores corresponding to three individual periods (i.e., 0.75, 0.8, and 0.85 seconds) are calculated. As shown in Table 2.22, superficial variation (i.e., $\pm$ 0.1) deviated from the structural score for the $T_e = 0.8$-second period is observed in all *seismic zones*. Therefore, 0.8 s is taken as the fundamental period of six-*storey* WLF and WPB *buildings* in the *building* height modifier calculation.

Table 2.22: Differences of structural scores due to variation of fundamental period of six-*storey* wood *buildings*

Seismic zone	WLF		WPB	
Very Low	0.1	-0.1	0.1	-0.1
Low	0.1	0	0	-0.1
Moderate	0.1	0	0.1	0
Moderately High	0.1	0	0	-0.1
High	0	-0.1	0.1	0
Very High	0.1	0	0	-0.1
Very High Max	0	0	0	0

The *building* height modifier does not apply to a few *model building types* because of *building* height restrictions as per the NBC. For example, SLF (steel light frame), PCF (precast frame), and MH (manufactures homes) *buildings* are typically three *storeys* or less, and hence *building* height modifiers are not calculated for these *buildings*. Similarly, *building* height modifiers for high-rise *buildings* do not apply to WLF (wood light frame), WPB (wood post-and-beam), and URM (unreinforced masonry bearing walls) *building type*s.

It is recognized that some older *buildings* were designed and constructed without height restrictions (e.g., brick *buildings* taller than seven *storeys*). Since there is no available date to develop *building* capacity and fragility curves for these *buildings*, *building* height modifiers for high-rise *buildings* cannot be calculated for them. Nevertheless, screeners may apply the *building* height modifier for mid-rise *buildings* to these *buildings*.

2.5.6 Building deterioration and age

Seismic risk screening implicitly assumes that a *building* has had regular and thorough maintenance throughout its service life, and that its seismic performance would not experience structural degradation/weakening caused by *deterioration* (Matsuki, et al. 2006, Tesfamariam 2008). In reality, however, operational, policy and budgetary priorities may limit the scope of *building* maintenance, and all structures are affected to varying degrees by normal *deterioration* and decay. *Building deterioration* decreases structural capacity and increases seismic vulnerability. Furthermore, *building deterioration* may result in additional *building irregularity* caused by changes in stiffness and strength. *Building* ageing can potentially affect the seismic performance of *buildings* as it may degrade the strength of structural elements, reducing the capacity of SFRS. Comparing to *building deterioration*, *building* ageing is a global effect and is unlikely to induce additional *building irregularity*. In this section, existing methods to assess *building deterioration* and age effects are introduced and discussed. The *building deterioration* and age modifier is then determined by adjusting the values of affected parameters in Table 2.12.

The manner in which *building deterioration* and *age* effect is addressed varies between seismic screening and evaluation methods. FEMA P-154 (2015) assumes that the *building* is constructed of sound materials. *Buildings* with significant *damage* or *deterioration* are candidates for detailed seismic evaluation. Because it is difficult to quantitatively assess the level of "significant," FEMA P-154 provided observable and additional guidance on identifying *damage* and *deterioration* for common *building* materials. However, for *buildings* with a level of *deterioration* less than "significant," no score modifier is applied to address its potential detrimental effect. OSHPD HAZUS (2010, 2016) considered the effect of *building deterioration* on structural overstrength and ductility capacity and hysteretic response. Comparing with the "significant" level of *deterioration* in FEMAP-154, the level of *deterioration* considered in OSHPD HAZUS is less severe. A *deterioration* score modifier is calculated based on the modified capacity and hysteretic response parameter values. In the 1993 NRC screening manual and a subsequent update by Saatcioglu et al. (2013), *deterioration* is explicitly treated as a type of *irregularity*; a factor of 1.3 (equivalent to vertical *irregularity*) is assigned to *deterioration* in the seismic screening. The level of *deterioration* considered is comparable to the "significant" level in FEMA P-154. Tischer et al. (2014) accounts for *deterioration* by empirically increasing seismic response spectra, thereby increasing seismic vulnerability in proportion to the estimated level of *deterioration*. For severe *deterioration* (referred to the *deterioration* level in the 1993 NRC screening manual), seismic demand is increased by 3.5 times accounting for the detrimental effect of *deterioration* on the *building's* seismic performance; for significant *deterioration* (i.e., *damage* or poor condition of visible structural elements), the seismic demand is increased by 1.5 times. Nevertheless, the rationale of increasing seismic demand is unclear because *building deterioration* is a *building* condition, disregarding the input seismic hazard. In "Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings," developed by the Japanese *Building* Disaster Prevention Association (JBDPA) (2001) – referred to as the Japanese seismic evaluation standard – a time index T is used to evaluate the effects of structural defects such as cracking, deflection, *ageing* and the like on a structure's seismic performance. For cracking in walls and columns, T is taken as 0.8 or 0.9 depending on the severity of the cracking; for *deterioration* due to occupation (e.g., chemical has been used), *building age*, and finishing condition, T is taken as 0.8 or 0.9 depending on the severity of the condition. In ASCE/SEI 41 (2013, 2017), a knowledge factor is used to reduce component capacity based on the level of knowledge and physical condition of the element. For example, for concrete components, capacities and deformations are calculated based on a knowledge factor of 0.75 if the component was found to be damaged or deteriorated. In the Indian "Guidelines for Seismic Evaluation and Strengthening of Buildings" by Rai (2005), a knowledge factor is also used to scale the strength capacity of *existing building* components based on the probable material strength and element condition. For example, a knowledge factor of 0.80 is recommended for a component with moderate *deterioration*.

Some conclusions are made based on reviewing the current seismic screening and evaluation methods:

- Not all studies differentiated *ageing* from *deterioration* in the *seismic risk* screening or evaluation. This may be attributed to the fact that *building deterioration* appears to interact with *ageing* during the *building's* service life, especially in older *buildings*.
- The level of *deterioration* such as "moderate" or "significant" in *seismic risk* screening is based on qualitative and subjective judgement. This is due to the fact that it is impossible to quantitatively evaluate the effect of *building deterioration* in a visual screening process.
- The 1993 NRC screening manual, Saatcioglu et al. (2013) and the Japanese seismic evaluation standard were developed based on qualitative approaches. Although they are not compatible with the quantitative methodology in the *Level 2 – SQST*, they provided valuable insights for assessing *building deterioration* and *ageing* effect. For example, the 1993 NRC screening manual and Saatcioglu *et al.* (2013) treated *building deterioration* as an *irregularity* whose severity is equivalent to vertical *irregularity*. In the first level of the Japanese seismic evaluation, time index T is specified for *building age*, i.e., $T = 1.0$ for *building age* less than 20 years, $T = 0.9$ for *building age* between 20 and 30 years, and $T = 0.8$ for *building age* 30 years or older.
- OSHPD HAZUS did consider *building deterioration* as a deficiency and provided a *deterioration* modifier, while ignoring the possible decay of *building* yielding capacity and displacement limit in seismic screening. In reality, localized *building deterioration* (e.g., corroded reinforcement of a critical element of SFRS) could affect the *building* yielding capacity and deformation capacity.
- ASCE/SEI 41 considered a knowledge factor of 0.75 to both material strength and permissible deformation. However, Rai (2005) applied a knowledge factor of 0.8 to material strength only, ignoring the possible decrease in *building* deformation capacity.

Based on the preceding discussion and comparison, a single *building deterioration* and age modifier is determined for moderate *building deterioration* and *ageing*. Buildings with significant deterioration or damage (e.g., rotted timber column, exposed reinforcing steel or large cracks in concrete shear walls) are flagged for *Level 3 – SEG*.

The *building deterioration* and age modifier for *building deterioration* considers two cases: (1) moderate *building deterioration* occurred in irregular *buildings*, and (2) moderate *building deterioration* occurred in regular *buildings*.

- For case 1, potential *building irregularity* induced by *building deterioration* is not considered because applicable *building irregularity* modifier has addressed *building irregularity*. A reduction factor of 0.75 that is equal to the knowledge factor in

ASCE/SEI 41-17 (2017) is applied to the yield capacity (Table 2.12) when developing the *building* capacity curve. The factor is based on available seismic assessment approaches (Saatcioglu et al. 2013; Tischer et al. 2014; JBDPA, 2001; ASCE 2017; Rai 2005) in which the reduction factor ranges from 0.7 to 0.9.

- For case 2, a reduction factor of 0.75 is applied to the yield capacity when developing the *building* capacity curve, and a *moderate vertical irregularity* is considered when generating the fragility curve and determining collapse factor (Table 2.12). A *moderate vertical irregularity* rather than severe vertical or horizontal *irregularity* is taken because the interstorey drift ratio reduction due to *moderate vertical irregularity* is 20% (equivalent to a reduction factor of 0.8) and is thus very close to the factor of 0.75 used in ASCE/SEI 41.

It is not practical to define a quantitative criterion for triggering *building deterioration*; therefore, the qualitative criteria presented in OSHPD HAZUS and FEMA P-154 are adopted in the structural scoring methodology.

The *building deterioration* and age modifier for *building* age is determined by applying a reduction factor of 0.8 to the yield capacity and using the value of degradation parameter associated with the *pre-code* design in accordance with FEMA P-154. The *building deterioration* and age modifier for *building* age is considered when *building* age is greater than 30 years. This number is based on existing seismic screening and evaluation guidelines (JBDPA 2001; NRC 2015; OSHPD 2016) in which the number ranges from 20 to about 90. The *Level 2 – SQST* project team decided not to choose the number 20 as the triggering criterion. A *building* occupied less than 20 years is judged to be relatively new, given normal maintenance. If 20 years was chosen as the criterion, even *buildings* built in 2000 would trigger the *building deterioration* and age modifier very early. Considering the limited decay of *building* capacity for a relatively new *building*, it is too conservative to apply the *building deterioration* and age modifier.

2.5.7 **Redundancy**

Redundancy is a beneficial *building* characteristic for preventing *building* collapse. A redundant structure is one in which the SFRS is distributed throughout the *building*, such that if *damage* were to occur to one or more elements, the remaining elements would provide the necessary strength to prevent collapse through load redistribution (FEMA P749 2010). *Redundancy* was first addressed in the 1997 UBC and a *redundancy* factor was assigned in the design for new *buildings*. As per ASCE/SEI 7 (2016), a *redundancy* factor shall be assigned to the SFRS in both orthogonal directions for all structures. The value of this factor is either 1.0 or 1.3. For structures assigned to Seismic Design Categories (SDCs) B and C, a *redundancy* factor is permitted to equal to 1.0. However, for structures assigned to SDCs D, E, and F, *redundancy* factor shall be equal to 1.3 for penalizing structures unless one of the following two conditions is met:

1. Each *storey* resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3 in the ASCE/SEI 7-16, which specifies that the removal of an

individual lateral force-resisting element would not result in more than a 33 percent reduction in *storey* strength, nor does the resulting system have an extreme torsional *irregularity*.

2. Structures that are regular in plan at all levels, provided that the SFRS consists of at least two bays of seismic force-resisting perimeter framing on each side of the structure, in each orthogonal direction, at each *storey* resisting more than 35 percent of the base shear.

A *redundancy* modifier is considered in the FEMA P-154 Level 2 optional screening and applied to *buildings* with at least two bays of lateral elements on each side of the *building,* in each direction. For *buildings* with shear walls, if the number of bays is not clear, then a bay can be defined as at least the height of the *storey* (FEMA P-154 2015). The *redundancy* modifier is determined based on the combination of the relative severity of *redundancy* against *severe vertical irregularity* (FEMA P-155 2015). The FEMA P-154 project team judged that the relative severity of *redundancy* is low; therefore, the *redundancy* modifier was taken as 33% of the average of Level 1 *severe vertical irregularity* modifiers for all FEMA *building* types (FEMA P-155 2015).

It is noticed that *redundancy* is not considered in Canadian seismic screening tools (NRC 1993; Karbassi and Nollet 2008; Saatcioglu et al. 2013; Tischer et al. 2014). The reason may be attributed to the fact that *redundancy* has never been explicitly addressed in the NBC. Considering the fact that redundant structures are expected to perform better when subject to earthquake ground motions, a *redundancy* modifier is determined for redundant *buildings* in the structural scoring methodology. Similar to FEMA P-154, the *redundancy* modifier is calculated as 33% of the average of *severe vertical irregularity* score modifiers for all *model building type*s used in the *Level 2 – SQST*. FEMA P-154 *redundancy* criteria are adopted to identify whether or not a *building* is redundant given the similarities in the seismic provisions in *building* codes and material design standards in Canada and the U.S.

2.5.8 **Pounding**

Pounding can occur if two *buildings* have significant differences in mass, stiffness, and strength, and there is insufficient separation distance between the two *buildings* to vibrate freely. For example, floor-to-column *pounding* can occur if floor levels in adjacent *buildings* are misaligned. This type of collision has the potential to cause large impact forces in the column, thus potentially leading to shear failure (Filiatrault et al. 1994). *Pounding* is also a major concern in dense urban areas, where *buildings* at the end of a row of closely spaced *buildings* are likely to experience greater *damage* since there are no adjacent *buildings* to resist collision forces (FEMA P-154 2015). Collision of adjacent structures can also cause *damage* to non-structural *building* components (Filiatrault and Cervantes 1995). In this section, existing methods for addressing the *pounding* effect are introduced and discussed. The *pounding* modifier is then determined by adapting the FEMA P-154 methodology to Canadian *seismicity* and *building* design practise.

In the 1993 NRC screening manual, *pounding* is considered to be a *building irregularity* that is triggered if the separation distance, d, between adjacent *buildings* is less than the threshold value calculated as follows:

$$d = 20\, Z_v n_S \text{ (in mm)} \tag{2.27}$$

where Z_v is the site-specific zonal velocity from the NBC 1990, and n_s is the number of *building storeys*.

Cole et al. (2012) observed that *buildings* subject to *pounding*, but without the following types of characteristics, are less likely to experience detrimental effects.

1. Floor-to-column;
2. Adjacent *buildings* with greatly differing mass;
3. *Buildings* with significantly different heights;
4. External (end) *buildings* in a row;
5. *Buildings* subject to plan torsion;
6. *Buildings* made of brittle materials.

In "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" developed by the New Zealand Society of Earthquake Engineering (NZSEE) (2006), *pounding* effect is considered in the Initial Evaluation Procedure (IEP) and the Detailed Seismic Assessment. Two *pounding* reduction factors D_1 and D_2 are applied to base factors for addressing Type 1 and Type 3 characteristics in Cole et al. (2012), as shown in Table 2.23. In the IEP, the base factor is equal to 1.0. The smaller the reduction factor, the more severe the *pounding*. For adjacent *buildings* with a gap less than the specified minimum separation ratio, but without Type 1 or Type 3 characteristics, *pounding* reduction factors are not applied. It is noted that "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" has been superseded by "The Seismic Assessment of Existing Buildings" in 2017 (NZSEE 2017). The method for evaluating the *pounding* effect remains unchanged.

Table 2.23: *Pounding* reduction factors in NZSEE (2006)

Pounding	Building characteristics	Severe	Significant	Insignificant
		$d/H < 0.5\%$	$d/H < 1\%$	$d/H > 1\%$
Type 1	(a) Alignment of floors within 20% of *storey* height	1.0	1.0	1.0
	(b) Alignment of Floors not within 20% of *storey* Height	0.4	0.7	0.8
Type 3	(a) Height difference < 2 *storeys*	1.0	1.0	1.0
	(b) Height difference 2 to 4 *storeys*	0.4	0.9	1.0
	(c) Height difference > 4 *storeys*	0.4	0.7	1.0

Tischer et al. (2014) quantified the severity of *pounding* based on the minimum separation ratios in NZSEE (2006). The modified structural scores are computed by increasing the seismic demand according to *pounding* effect severity. For example, a factor of 1.5 was applied to the seismic demand for significant *pounding* identified as Type 1 (b) in Table 2.23.

FEMA P-154 (2015) considered the *pounding* in Level 2 optional screening if two adjacent *buildings* are separated by less than the specified separation distance d (Table 2.24), and one of the following three conditions are present: (i) floors do not align vertically within two feet, which is equivalent to Type 1 (b) in NZSEE (2006), given a typical *storey* height of three metres; (ii) one structure is more than two *storeys* taller than the other, which is equivalent to the sum of Type 3 (a) and 3 (b) in NZSEE (2006); and (iii) the *building* is at the end of a block (Type 4).

Table 2.24: Recommended minimum separation distances for preventing *pounding* in FEMA P-154 (2015)

Seismicity region	Minimum separation ratio d
Low	0.10% H
Moderate	0.25% H
Moderately High	0.50% H
High	1.00% H
Very High	1.50% H

Comparing with NZSEE (2006), one additional *pounding* type, Type 4, is considered. Nevertheless, comparing with Cole et al. (2012), significant mass difference (Type 2), plan torsion (Type 5), and brittle materials (Type 6) are not addressed. This may be attributed to the fact that it is difficult to assess these three types in a rapid visual screening process. Similar to the approach for determining the *redundancy* modifier, a *pounding* modifier is determined based on the relative severity of the *pounding* against the *severe vertical irregularity*: (i) 100% of the average of the Level 1 *severe vertical irregularity* modifiers for all FEMA *building type*s is assigned to conditions (i) and (ii), while 50% of the average is assigned to condition (iii). When more than one of the *pounding* types are present, the applicable *pounding* modifiers are summed, but should not exceed the cap indicated on the Level 2 optional scoring form (FEMA P-154 2015). For adjacent *buildings* with a gap less than the recommended minimum separation distance in Table 2.24, but without any one of the three conditions considered in the scoring procedure, no *pounding* modifier is applied.

The *Level 2 – SQST* project team carefully examined the separation limits in the 1993 NRC screening manual, NZSEE (2006) and FEMA P-154 (2015) to determine their applicability in the structural scoring methodology. Since *building* response is a function of the *building's* fundamental period and ground shaking intensity, it is reasonable that the minimum separation distance d would be influenced by these parameters. The 1993 NRC screening manual acknowledged the influence of ground shaking intensity only. In addition, the limits in the 1993 NRC screening manual were defined as a function of zonal velocity Z_v that has been dropped since the NBC 2005. Therefore, the 1993 NRC screening manual was deemed no longer applicable for determining the *pounding* modifier in the structural scoring methodology. NZSEE does not consider *seismicity* in its assessment of *pounding*, and was hence removed from consideration. FEMA P-154 (2015) acknowledged both *building* fundamental period and ground shaking intensity, and considered three *pounding* types. Therefore, the *Level 2 – SQST* project team decided to adopt the FEMA P-154 methodology to determine the *pounding* modifier.

Although FEMA P-154 (2015) provided the minimum separation distances as shown in Table 2.24, these values were judged not to apply to the *Level 2 – SQST* since they were developed based on U.S. *seismicity*. Given this, the minimum separation distances in FEMA P-154 are adapted to suit Canadian *seismicity* by utilizing the FEMA P-154 methodology. The following discussion on calculating theoretical separation distance limits is based primarily on the procedure described in Chapter 6 of FEMA P-155 (2015).

The separation distance d between two *buildings* is computed in a simplified manner assuming that two *buildings* with the same height H and fundamental period T, experience their maximum deflection u_{max} at the same time. *Pounding* will occur if the separation distance d between the *buildings* is some fraction less than $2.0 \times u_{max}$. The peak displacement u_{max} of each *building* can be calculated from a response spectrum analysis (Chopra 2007) for acceleration- and velocity-controlled response according to:

$$u_{max} = \begin{cases} \dfrac{T^2}{\alpha_2} \cdot \dfrac{\mathrm{Max}[S_a(0.2), S_a(0.5)]}{4\pi^2} & \text{for } T < T_s \\ \dfrac{T}{\alpha_2} \cdot \dfrac{S_a(1.0)}{4\pi^2} & \text{for } T \geq T_s \end{cases} \tag{2.28}$$

where: $S_a(0.2)$, $S_a(0.5)$, and $S_a(1.0)$ are spectral accelerations at 0.2-, 0.5- and 1.0-second periods, respectively; T is the *building's* fundamental period; α_2 is the modal height factor used to convert the *building* to an equivalent SDOF system; and T_S is the transition period between acceleration- and velocity-controlled responses.

The peak displacement, u_{max}, is calculated for five generalized *building type*s as per the NBC 2015 in a 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 Section 2.5.5, 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 fundamental periods T for the five generalized *building type*s are computed in accordance with Article 4.1.8.1 in the NBC 2015, representing the typical range of stiffness and mass for each type of SFRS (note that n_s for Type 3 *buildings* below refers to the number of *storeys*):

$T = 0.085\ (H)^{3/4}$	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)
$T = 0.025\ H$	For braced frames (Type 4)
$T = 0.05\ (H)^{3/4}$	For shear walls and other structures (Type 5)

The modal height factor α_2 in Eq. (2.28) is the inverse of the modal participation factor Γ and is a measurement of the degree to which the considered mode contributes to response (Chopra 2007). Values of α_2 are obtained from APPENDIX D corresponding to 3, 7, and 15 *storeys*. Specifically, for low-rise *buildings*, $\alpha_2 = 0.75$; for mid-rise *buildings*, $\alpha_2 = 0.69$; for high-rise *buildings*, $\alpha_2 = 0.60$. Values of $S_a(0.2)$, $S_a(0.5)$, and $S_a(1.0)$ used to calculate u_{max} are from Table 2.10 for Very Low through Very High *seismic zones* for reference *Site Class* C.

The minimum separation distance d between two adjacent *buildings* is determined by using the square root of sum of squares (SRSS) combination rule, in accordance with the NBC 2015, giving a minimum separation distance $d \approx 1.41u_{max}$. It is noted that in FEMA P-154 the minimum separation distance d between two adjacent *buildings* is taken as u_{max}, instead of using the SRSS combination rule that is adopted in the ASCE/SEI 41-17 and IBC 2018. The FEMA P-154 project team demonstrated that:

"*Using an SRSS combination of the u_{max} values would make sense if they represented the simultaneous maximum displacement of an object in two orthogonal directions and the important issue was the maximum absolute displacement regardless of direction, but that is not what is occurring here. Using an SRSS combination obscures the assumptions of timing and phase difference rather than reflecting a physical reality.*"

The structural separation ratio, d/H, expressed as a percentage, is introduced to normalize the separation distances, d, to the minimum height of adjacent *buildings*, H. The minimum separation ratio d/H is calculated for each *seismic zone*, *building* height, and generalized *building type.* The results are provided in Table 2.25. It is seen that the separation ratio does not exhibit significant variation within each *building type* (except low-rise Type 3 and Type 4 *buildings*) as a function of height. Therefore, for Type 1, Type 2, and Type 5 *buildings*, the values of d/H are taken as the averaged values for low-, mid-, and high-rise; for Type 3 and Type 4 *buildings*, since the values of d/H for low-rise *buildings* are significantly lower than those for mid- and high-rise *buildings*, the values of d/H are taken as the averaged values for mid- and high-rise *buildings*, preventing values of d/H from being underestimated. Table 2.26 shows the structural separation ratio d/H as a function of *seismic zone* and *building type*.

Table 2.25: Separation ratio as a function of *seismicity*, *building* height, and *building type*

Seismicity	**Height**	**d/H (%)**				
		Type 1	**Type 2**	**Type 3**	**Type 4**	**Type 5**
Very Low	low-rise	0.06	0.06	0.02	0.02	0.03
	mid-rise	0.06	0.05	0.04	0.04	0.03
	high-rise	0.05	0.05	0.04	0.04	0.03
Low	low-rise	0.15	0.13	0.05	0.05	0.06
	mid-rise	0.13	0.12	0.09	0.09	0.08
	high-rise	0.13	0.11	0.10	0.10	0.07
Moderate	low-rise	0.26	0.23	0.09	0.09	0.11
	mid-rise	0.23	0.20	0.15	0.15	0.13
	high-rise	0.22	0.19	0.18	0.18	0.13
Moderately High	low-rise	0.43	0.38	0.18	0.18	0.21
	mid-rise	0.38	0.33	0.25	0.25	0.22
	high-rise	0.36	0.32	0.29	0.29	0.21
High	low-rise	0.88	0.77	0.30	0.30	0.35
	mid-rise	0.77	0.68	0.52	0.52	0.45
	high-rise	0.73	0.65	0.60	0.60	0.43
Very High	low-rise	1.49	1.27	0.49	0.49	0.57
	mid-rise	1.32	1.16	0.89	0.89	0.77
	high-rise	1.25	1.11	1.03	1.03	0.74
Very High Max	low-rise	1.92	1.58	0.61	0.61	0.70
	mid-rise	1.69	1.49	0.15	0.15	0.10
	high-rise	1.61	1.42	1.32	1.32	0.95

Table 2.26: Separation ratio as a function of *seismicity* and *building type*

Seismicity	d/H (%)					
	Type 1	Type 2	Type 3	Type 4	Type 5	Average
Very Low	0.06	0.05	0.04	0.04	0.03	0.04
Low	0.14	0.12	0.10	0.10	0.07	0.10
Moderate	0.23	0.20	0.14	0.14	0.12	0.17
Moderately High	0.39	0.34	0.27	0.27	0.21	0.30
High	0.79	0.70	0.56	0.56	0.41	0.61
Very High	1.35	1.17	0.81	0.81	0.69	1.02
Very High Max	1.74	1.50	1.23	1.23	0.88	1.32

Although the d/H ratio varies with *building type*, a specific d/H ratio is recommended for each *seismic zone*, which is consistent with FEMA P-154. Table 2.27 presents the recommended d/H ratios for all *model building types* in each *seismic zone*. It is noted that the d/H ratio for Very High Max is not provided because Very High Max represents the maximum *seismicity* (i.e., extreme end of the Very High *seismic zone*) in Canada. Compared to the average d/H ratios in Table 2.26, the recommended ratios are conservative for the majority of *model building types*.

Table 2.27: Recommended *d/H* ratio as a function of *seismicity*

Seismicity	VL	L	M	MH	H	VH
d/H (%)	0.05	0.1	0.2	0.4	0.8	1.2

Following the FEMA P-154 methodology, the *pounding* modifier is determined based on the relative severity of *pounding type* against *severe vertical irregularity* (Table 2.28). *Pounding* is triggered as long as the separation ratio d/H is less than the thresholds in Table 2.27. Although a *building* structure may not experience severe *damage* without the specified *pounding* types in FEMA P-154, non-structural components in the *building* may be *damage*d due to collision forces of adjacent *buildings* and pose life safety hazard to the *occupants* and pedestrians (Filiatrault and Cervantes 1995). Given this, an additional "other" is considered in the structural scoring methodology for *pounding* types that are not classified as any one of the three specified *pounding* types. The relative severity of the "other" *pounding* types is judged to be low by the *Level 2 – SQST* project team; therefore, the *pounding* modifier for this *pounding* type is taken as 33% of the average of *severe vertical irregularity* modifiers for all *model building type*s.

Table 2.28: Determination of *pounding* modifiers in the *Level 2 – SQST*

Pounding type	Relative severity	Pounding modifier
Other types not listed below	Low	33% of the average of *severe vertical irregularity* modifiers for all *model building types*
Building is at the end of the block	Moderate	50% of the average of *severe vertical irregularity* modifiers for all *model building types*
Floors not aligning vertically within 600 mm	High	100% of the average of *severe vertical irregularity* modifiers for all *model building types*
Building height differences greater than two *storeys*	High	100% of the average of *severe vertical irregularity* modifiers for all *model building types*

2.5.9 Remaining occupancy time

An *existing building* is expected to have a shorter *remaining occupancy time* than a new *building*, and therefore has a smaller chance of experiencing a code level earthquake event over its remaining *occupancy* (ASCE/SEI 41 2017). Therefore, considering the effect of *remaining occupancy time* in the seismic screening provides a more accurate assessment of the seismic vulnerability of an *existing building* if the *building* owner or local *building* department confirms a shorter *remaining occupancy time*.

If a *building* is designed for an earthquake response spectrum with a constant probability of exceedance over its design service life (e.g., 50 years as per the NBC), it becomes increasingly less likely that the *building* will experience a code level earthquake as it approaches the end of its service life. Fahti-Fazl et al. (2018) proposed a *remaining occupancy time* factor κ 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 determine 5%-damped response spectra corresponding to return periods other than 2475 years (i.e. return period of code level earthquake) as follows:

$$S(T)_{2\%/n} = \kappa \cdot S(T)_{2\%/50} \tag{2.29}$$

where $S(T)_{2\%/50}$ is the spectral acceleration of a code level earthquake as per the NBC 2015, $S(T)_{2\%/n}$ is the reduced spectral acceleration with a probability of exceedance of 2% in n years, and κ can be calculated as:

$$\kappa = 1.133 - 1.05e^{-0.041n} \tag{2.30}$$

The Eq. (2.30) was obtained from regression analyses for 679 locations provided in Appendix C of Division B of the NBC 2015 (Fahti-Fazl et al. 2018). The κ values for 5, 10, 15, 20, 25 and 30 years are equal to 0.28, 0.44, 0.57, 0.67, 0.76 and 0.83, respectively.

A *remaining occupancy time* modifier is used to account for the reduction in response spectrum corresponding to code level earthquake as a function of *remaining occupancy time*. The *remaining occupancy time* modifier values are calculated by scaling the spectral accelerations for *Site Class* C in Table 2.10 by κ values corresponding to 5, 10, 15, 20, 25 and 30 years, respectively. The other parameter values in Table 2.12 are taken as those for calculating the structural basic scores. The *remaining occupancy time* modifier values are positive, indicating that the *remaining occupancy time* is a beneficial *building* condition in *seismic risk* screening.

Providing a positive score modifier for *buildings* with shorter *remaining occupancy time* is justified since there is a corresponding reduction in *seismic risk* due to a decrease in the likelihood of experiencing a code level earthquake. However, identifying *remaining occupancy time* must be fairly rigorous or it is open to abuse, i.e., *buildings* that remain on the cusp of decommissioning or divestiture for well over a decade. Using a suspected life span is not sufficient to address due diligence in a *building* that is otherwise questionable in terms of seismic resistance. The *remaining occupancy time* modifier should only be applied when there exists a signed document from the owner/investor to the effect that the plan for the *building* is officially to decommission it or sell it within a prescribed period of time. If there is any ambiguity or uncertainty regarding the *remaining occupancy time*, the *occupancy time* modifier should not be applied and the structural scoring should be based on the full design response spectrum with a 2% probability of exceedance in 50 years. If a *building* has been screened on the basis of *remaining occupancy time* for a particular tenant, and a new tenant arrives and prolongs the *building's remaining occupancy time*, then a subsequent re-screening should be conducted based on the new tenant's planned *occupancy*.

Although the *remaining occupancy time* modifier is calculated for a *remaining occupancy time* more than 10 years, in *Level 2 – SQST* screening forms, this modifier is provided only for *remaining occupancy time* of less than or equal to 10 years, considering the fact that the owner of the *building* is not expected to plan the *building's* decommission when its *remaining occupancy time* is relatively long, such as 15 years.

2.5.10 **Seismic upgrading**

Historically, the terms "seismic rehabilitation," "seismic retrofit," and "*seismic upgrading*" were used in different guidelines and standards to describe the systematic process to improve seismic resistance of *existing buildings* in order to make them meet current seismic evaluation provisions. To be consistent with the terminology in Commentary L of Structural Commentaries in the NBC 2015, the expression "*seismic upgrading*" is used in the structural scoring methodology.

Observations from previous major earthquakes around the world have shown that *existing buildings* designed to early seismic code editions, especially *pre-code buildings*, are prone to experiencing severe damage and even collapse when subject to strong earthquake shakings. In past decades, a variety of seismic evaluation and upgrading methodologies and techniques have been developed and used to mitigate the *seismic risk* of *existing buildings*. In this section, existing seismic evaluation and upgrading guidelines in the U.S. and Canada are introduced first. The *seismic upgrading* modifier is then determined by adapting the FEMA P-154 methodology to Canadian *seismic upgrading* practices.

In 1992, FEMA published the "NEHRP Handbook for the Seismic Evaluation of Existing Buildings" (FEMA 178 1992). The evaluation criteria are based on a reduction factor of 0.67 for most *buildings*, and 0.85 for stiff low-rise *buildings*. The FEMA 178 approach is retained in the IBC 2015, which allows a factor of 0.75 on seismic demand. The ASCE/SEI 41 (2017) achieved approximately the same effect by increasing component factors (i.e., *m* factors) in the Tier 2 procedure and by applying a 0.75 factor to the code-based seismic demands in its Tier 3 procedure. Many jurisdictions in the U.S., including Los Angeles and San Francisco, have used the 0.75 reduction factor for decades.

The 1993 NRC Guidelines for Seismic Evaluation of Existing Buildings customized FEMA 178 (1992) with modifications to suit Canadian seismic design and evaluation practices. A reduction factor of 0.60 rather than 0.75 was applied to the NBC seismic loading criteria, which corresponds to a U-factor of 0.36, as a minimum for triggering *seismic upgrading* for any deficiency. This reduction factor was chosen on the basis of the following considerations:

- The reduction factor of 0.75 was justified in the FEMA 178 as the removal of a hidden safety factor contained in the dynamic response factor (equivalent to *S*-factor in the NBC) associated with the use of a response spectrum in the NEHRP model code corresponding to the mean-plus-one standard deviation rather than the mean. Further background to this is given in the ATC-14 document entitled, "Evaluating the Seismic Resistance of Existing Buildings" (ATC 1987).
- A study by Allen (1991) on minimum load factors for structural evaluation of *existing buildings* based on the life-safety goal of the NBC recommends that the NBC-specified earthquake load be reduced as a function of the consequences of potential failure. The consequences are assessed on the basis of *redundancy* and the likelihood and number of people at risk (life-risk category). Assuming a "medium" *redundancy* as recommended by the 1995 NRC Guideline for Seismic Upgrading of Building Structures and a "normal" life-risk category, the risk study (Allen 1991) determined a reduction factor close to 0.6. The evaluator may wish to consider adjusting the 0.6 factor up or down, according to the *redundancy* and life-risk category for each potential failure.
- The resulting triggering criteria in the 1993 NRC evaluation guidelines were compared to those contained in FEMA 178 for the same *seismicity* (same ground velocity or acceleration), taking into account all factors entering into the determination of factored base shear. With a U-factor of 0.36, the 1993 NRC evaluation guidelines criteria range from a minimum of 0.9 to a maximum of 2.0 times the FEMA 178 criteria. Thus, the 1993 NRC evaluation guidelines criteria remain conservative in comparison with the FEMA 178 criteria.

In 1997, FEMA published "NEHRP Guidelines for the Seismic Rehabilitation of Buildings," providing technically sound and nationally acceptable guidelines for the seismic rehabilitation of *buildings*. A pre-standard based on this document was issued in 2000 (FEMA 356 2000) and was converted to a standard in 2006 (ASCE/SEI 41 2006). The standard was updated in 2013 as the "Seismic Evaluation and Retrofit of Existing Buildings" and superseded ASCE/SEI 41 (2006). It was most recently updated in 2017. Conventional and innovative techniques have been applied in the last three decades to make those seismically vulnerable *buildings* more resistant to earthquake effects (FEMA 172 1992; FEMA 547 2006).

In 1995, the NRC developed the "Guideline for Seismic Upgrading of Building Structures," to help engineers design *seismic upgrades* using appropriate techniques for correcting seismic deficiencies identified using the NRC 1993 evaluation guidelines. The 1995 NRC upgrading guideline was based in part on the NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings (FEMA 172 1992).

The seismic evaluation and upgrading of *Existing Buildings* were first addressed in Commentary K of Structural Commentaries in the NBC 1995, referencing the 1993 evaluation guidelines and 1995 upgrading guideline. These two technical guidelines were also referenced in the NBC 2005 and NBC 2010. Beginning with the NBC 2015, new procedures have been recommended for minimum voluntary seismic evaluation and upgrading, and minor/major renovations, as well as minor/major horizontal/vertical additions, compared with its previous editions. Site-specific seismic hazards correspond to four different levels of probability of exceedance (POE) in 50 years (Level 1: 0.5 times seismic hazard with a POE of 5% in 50 years, Level 2: seismic hazard with a POE of 10% in 50 years, Level 3: seismic hazard with a POE of 5% in 50 years, and code level seismic hazard with a POE of 2% in 50 years) are suggested in the seismic assessment and upgrading of *existing buildings* (NRC 2015).

For *buildings* that have undergone a comprehensive *seismic upgrading* (i.e., the entire SFRS being strengthened), some credits should be given in the seismic screening process to reflect the improved seismic resistance. FEMA P-154 (2015) considered a positive retrofit modifier in the Level 2 optional screening, provided that a comprehensive retrofit is recognized. Partial retrofits and in-progress or incremental retrofits are not provided with the modifier. If the retrofit appears to effectively counteract an observed deficiency, neither the deficiency nor the retrofit modifier should be applied. Similar to the approach for determining *redundancy* and *pounding* modifiers, the retrofit modifier is determined based on the relative severity of the retrofit condition against the post-benchmark design. Because the FEMA P-154 project team deemed the retrofit to be moderately high, 75% of the average of post-benchmark modifiers for all FEMA *building type*s was taken as the retrofit modifier. Considering that applicable retrofit standards, e.g., ASCE/SEI 41, used 75% of code level seismic forces when evaluating seismic force-resisting elements, selecting 75% of the average is judged to be reasonable.

Following the FEMA P-154 methodology, *seismic upgrading* modifiers were determined for the following cases:

- Case 1: *pre-code* or *pre-benchmark buildings* seismically upgraded to meet 100% of the seismic loading required in an applicable *benchmark NBC edition*. The seismic upgrading modifier is taken as the average of the *building* design period modifiers for *post-benchmark buildings*.

- Case 2: *pre-code* or *pre-benchmark buildings* seismically upgraded to meet not less than 60% but less than 100% of the seismic loading required in an applicable *benchmark NBC edition*. The seismic upgrading modifier is taken as 75% of average of *building* design period modifiers for *post-benchmark buildings*. This case is equivalent to the retrofit condition considered in FEMA P-154.

- Case 3: *pre-code buildings* seismically upgraded to satisfy at least 60% of the seismic loading required in an applicable *pre-benchmark NBC edition*. Since these *buildings* have been upgraded to an applicable *pre-benchmark code edition*, a seismic upgrading modifier should be provided to cancel out the *building* design period modifier for *pre-code buildings*. Given this, a positive value that is equal to the *building* design period modifier for *pre-code buildings* in magnitude is taken as the seismic upgrading modifier.

- Case 4: *pre-benchmark buildings* seismically upgraded to satisfy at least 60% of the seismic loading required in an applicable *pre-benchmark code edition*. Although seismic upgrading has been done, the *buildings* are still within the same *building* design period, namely pre-benchmark design period. As a result, a seismic upgrading modifier is not provided for this case.

Other cases such as partial *seismic upgrading* and in-progress or incremental *seismic upgrading* are not recognized and thus not provided with *seismic upgrading* modifiers.

Comparing with the retrofit condition in FEMA P-154 (equivalent to Case 2 in the *Level 2 – SQST*), three additional *seismic upgrading* cases (i.e. Cases 1, 3, and 4) are considered in the structural scoring methodology in order to capture all possible comprehensive *seismic upgrading* conditions.

2.5.11 **Seismic hazard**

Recall that, in the early editions of the UBC and NBC, *seismic zones* were defined in terms of peak ground velocity – PGV or peak ground acceleration – PGA. Given a *seismic zone*, the mean (or medium) seismic hazard rather than site-specific seismic hazard was used to calculate seismic demand of *buildings* within the *seismic zone*. However, due to the inherent variability of seismic hazard within a specified *seismic zone*, the seismic demand of a specific *building* may be lower or higher than the calculated seismic demand corresponding to mean (or medium) seismic hazard in the *seismic zone*. As a result, the seismic demand of *buildings* with seismic hazard lower than mean (or medium) could be overestimated, and the seismic demand of *buildings* with seismic hazard larger than mean (or medium) could be underestimated. Because the variation of the seismic demand can influence the *building's seismic risk* estimate, a seismic hazard modifier is considered to achieve more accurate *seismic risk* estimates of *existing buildings*.

Note that, in the *Level 2 – SQST*, mean seismic hazard of each *seismic zone* was used to calculate structural basic scores of *existing buildings*. To investigate the effect of prescribed seismic hazard on the structural basic score, structural basic scores S_B are calculated based on lower, mean, and upper threshold values of each *seismic zone*. The results are provided in Table 2.29. The S_B values for the lower bound threshold of the Very Low (VL) *seismic zone* are not provided because there is no lower bound threshold of seismic hazard provided for this *seismic zone*. The S_B values for Very High Max (VHX) are not presented because Very High Max represents the maximum *seismicity* in Canada and thus does not have statistical meaning.

Table 2.29: Structural basic scores corresponding to lower bound, mean, and upper bound seismic hazard in each *seismic zone*

Seismicity	Hazard	WLF	WPB	SMF	SBF	SLF	SCW	SIW	CMF	CSW	CIW	PCW	PCF	RML	RMC	URM	MH
Very Low	Mean	10.4	10.4	7.3	7.7	8.0	8.3	8.0	6.7	8.5	7.5	8.0	7.4	8.1	8.1	7.4	8.5
	Upper bound	9.3	9.3	6.3	6.7	7.0	7.3	7.0	5.8	7.5	6.6	7.0	6.5	7.2	7.2	6.5	7.5
Low	Lower Bound	9.3	9.3	6.3	6.7	7.0	7.3	7.0	5.8	7.5	6.6	7.0	6.5	7.2	7.2	6.5	7.5
	Mean	8.4	8.4	5.6	5.9	6.2	6.5	6.2	5.1	6.6	5.8	6.2	5.7	6.3	6.3	5.7	6.7
	Upper bound	7.3	7.3	4.8	5.1	5.3	5.5	5.2	4.3	5.7	4.9	5.3	4.8	5.3	5.3	4.8	5.9
Moderate	Lower Bound	7.3	7.3	4.8	5.1	5.3	5.5	5.2	4.3	5.7	4.9	5.3	4.8	5.3	5.3	4.8	6.4
	Mean	6.7	6.6	4.4	4.5	4.8	4.9	4.6	3.8	4.9	4.1	4.6	4.0	4.5	4.5	3.9	5.6
	Upper bound	5.8	5.6	3.9	4.1	4.3	4.1	3.8	3.2	4.0	3.3	3.8	3.2	3.6	3.6	3.1	4.5
Moderately High	Lower Bound	6.1	6.0	4.1	4.3	4.5	4.7	4.0	3.4	4.7	3.6	4.5	3.7	4.4	4.4	3.4	4.8
	Mean	5.3	5.2	3.6	3.8	3.9	4.0	3.3	2.9	3.9	3.0	3.8	3.1	3.7	3.7	2.8	4.1
	Upper bound	4.3	4.1	3.0	3.1	3.3	3.2	2.5	2.3	3.0	2.2	2.8	2.3	2.7	2.7	1.9	3.2
High	Lower Bound	4.4	4.3	3.1	3.3	3.4	3.4	2.5	2.4	3.3	2.2	3.1	2.4	3.0	3.0	1.9	3.2
	Mean	4.0	3.8	2.7	2.8	3.1	2.9	2.0	2.0	2.7	1.7	2.5	2.0	2.5	2.5	1.5	2.5
	Upper bound	3.4	3.2	2.5	2.5	2.7	2.5	1.7	1.8	2.3	1.4	2.2	1.7	2.1	2.1	1.3	2.2
Very High	Lower Bound	3.4	3.2	2.5	2.5	2.7	2.5	1.7	1.8	2.3	1.4	2.2	1.7	2.1	2.1	1.3	2.2
	Mean	2.8	2.6	2.0	2.0	2.2	2.0	1.4	1.4	1.8	1.2	1.6	1.3	1.6	1.6	1.0	1.8
	Upper bound	2.4	2.2	1.8	1.7	1.9	1.7	1.3	1.2	1.5	1.0	1.3	1.1	1.4	1.4	0.9	1.6

It is noticed that S_B values are not continuous at thresholds between the Moderate (M), Moderately High (MH), and High (H) *seismic zones*. This is due to the fact that parameter values used to develop *building* capacity curves and fragility curves are different depending on design code levels. The discontinuity of S_B values is reasonable since seismic design/detailing requirements that affect the ductility capacity and drift capacity are dependent on *seismic zones*. The S_B values are continuous in other boundaries because the parameter values used to develop *building* capacity curves and fragility curves are taken as the same in two adjacent *seismic zones,* based on the mapping of *seismic zones* and design code levels. An exception is the S_B value of manufactured homes (MH) between Low (L) and Moderate (M) *seismic zones*. This is because the values of medium interstorey drift ratio Δ_c for MH are different in these two *seismic zones*.

Although S_B values for more refined seismic hazard in each *seismic zone* are not provided in Table 2.29, it is found that S_B values are almost linear between two adjacent points (i.e., between the S_B value at threshold value and at mean) in each *seismic zone*.

Optionally, in the *Level 2 – SQST*, screeners may adjust the S_B values by linear interpolation between two adjacent points given a site-specific seismic hazard. The general procedure to interpolate the S_B values for site-specific seismic hazard is presented as follows:

1. Determine the *seismic zone* where the *building* is located and the *model building type*.
2. Determine the S_B value using the mean seismic hazard in the *seismic zone* (Table 2.10).
3. Calculate the adjusted S_B value by:
    a. interpolating $S_a(0.2)$ between the mean and the threshold value (upper bound or lower bound) provided in Table 2.10, or
    b. interpolating $S_a(1.0)$ between the mean and the threshold value (upper bound or lower bound) provided in Table 2.10.
4. Take the lower value of adjusted S_B as the final adjusted S_B value.

The seismic hazard modifier is determined by subtracting the adjusted S_B value by the S_B value based on the mean seismic hazard. The seismic hazard modifier does not apply to *buildings* designed and constructed to the NBC 2005 and its later editions, because in these editions, *seismic zones* were dropped and site-specific seismic hazards were used in the seismic design of *buildings*.

Compared to other structural score modifiers, the application of a seismic hazard modifier is not a requirement of the *Level 2 – SQST*. The *building* owner may apply this score modifier in accordance with the requirements and limitations in this subsection.

2.6 Acceptable structural thresholds

Determining an acceptable structural threshold is one of the most difficult issues pertaining to quantitative *seismic risk* screening, which involves a compromise between the benefits of increased safety, and the cost to ensure that level of safety (FEMA 154 2002). In this section, the determination of cut-off score (i.e. acceptable structural threshold) in FEMA P-154 is presented first. The acceptable structural thresholds in the structural scoring methodology are then determined in accordance with the FEMA P-154 approach, with adjustment to consider different levels of *consequences of failure* in the *Level 2 – SQST*.

2.6.1 Determination of cut-off score in FEMA P-154

In FEMA P-154, final score *S* estimates the conditional probability of collapse, $P(COL|MCE_R)$, given the risk-targeted maximum considered earthquake (MCE_R) corresponding to a specified *seismicity* region. A *building* may experience many earthquakes during its service life and almost none of them have shaking exactly equal to the MCE_R shaking. However, the score *S* cannot deal with the probability of collapse condition on other ground shaking levels or the chance of a collapse-causing earthquake during the design life, because it relates only the collapse probability to a given MCE_R shaking (page 8-2 in FEMA P-155 2015). Given this, FEMA P-155 introduced risk score S_R to measure *building* safety in terms of how frequently collapse-causing earthquake occur. It is defined as the negative common logarithm of the number of earthquakes that could cause *building* collapse during a *building's* design life (commonly taken as 50 years). Risk modification factor PMF_R is used to relate S to S_R:

$$PMF_R = S_R - S \tag{2.31}$$

Table 2.30 provides the expected values of PMF_R for each *seismicity* region (Table 8-3 in FEMA P-155 2015). It shows that, for Moderate through Very High *seismicity* regions, the probability of collapse in the design life is approximately equal to 0.1 ($PMF_R \approx 1.0$) times the conditional probability of collapse given MCE_R shaking (i.e. $P[COL|MCE_R]$). In other words, the ratio of probability of collapse to conditional probability of collapse is almost a constant, disregarding the level of *seismicity*.

Table 2.30: Expected values of risk modification factor

Seismicity region		PMF_R
	Low	0.1
	Moderate	0.9
	Moderately High	1.2
	High	1.1
	Very High	0.9

Therefore, one can add 1.0 to final score S to get risk score S_R (FEMA P-155 2015):

$$S_R = S + 1 \tag{2.32}$$

The S_R score is useful because one can relate the final score S to fatality risk. Table 2.31 presents fatality risk of *existing buildings* compared with new *buildings* as a function of S_R. An S_R value of 3.5 (i.e., S=2.5 based on Eq. (2.32)) is equivalent to the fatality risk for a new *building*.

Table 2.31: Fatality risk of *existing buildings* compared with new *buildings* (from FEMA P-155 2015)

S_R	Fatality Risk Multiplier of Existing Buildings Compared with New Buildings
1.5	100×
2.0	32×
2.5	10×
3.0	3×
3.5	1×
4.0	0.3×
4.5	0.1×

Assuming that *existing buildings* have a somewhat lower value of S than new *buildings*, FEMA P-154 selected a cut-off score of S_{TH}=2.0 (equivalent to S_R=3.0) for ordinary (Risk Category II) *existing buildings*. S_{TH}=2.0 can be also obtained from the following Eqns. (2.33) and (2.34), using a 1% probability of failure as in ASCE/SEI 7-16 for Risk Category II *buildings* and a collapse portion of 0.1 used to equate the probability of collapse in FEMA P-154 and the probability of failure in ASCE/SEI 7-16 (pages 4-8 and 4-10 in FEMA P-155 2015):

$$S_R = -\log_{10}(1\% \times 0.1) = 3 \tag{2.33}$$

$$S_{\mathrm{TH}} = S_R - 1 = 2 \tag{2.34}$$

Because $\log_{10}(\text{collapse portion} = 0.1) = -1$, the calculation of the S_{TH} can be simplified as

$$S_{\mathrm{TH}} = -\log_{10}(\text{probability of failure in design life}) = -\log_{10}(1\%) = 2 \tag{2.35}$$

2.6.2 Determination of acceptable structural thresholds

Building failure can pose different levels of consequences associated with the risk to human lives. The NBC 2015 addresses different levels of consequence of disruption in use of *buildings* by considering *Low*, *Normal*, *High*, and *Post-disaster* importance categories. The increase in importance category results in an increase in *building* capacity, which in turn reduces the probability of *building* failure. However, the importance categories do not consider other applicable key parameters affecting *buildings' consequences of failure* such as *building occupancy* and *building* size. For example, a one-storey office *building* and a twenty-storey office *building* may both be classified as *Normal importance* category; however, the *consequences of failure* of these two *buildings* are significantly different because the taller office *building* can accommodate far more *occupants* than the one-story *building* does. To address this, the consequence classification system proposed by Fathi-Fazl and Lounis (2017a) was adopted to describe different levels of *consequences of failure*, i.e., Very Low (VLC), Low (LC), Medium (MC), High (HC) and Very High (VHC). The instructions for identifying the *consequences of failure* for different *building occupancy* types are provided in Part 1: User's Guide of the *Level 2 – SQST*.

It is noticed that Risk Categories in ASCE/SEI 7-16 are defined based on the seriousness of the *consequences of failure* of *buildings*, which are comparable to different levels of *consequences of failure* in the *Level 2 – SQST*. A mapping between Risk Categories and *consequences of failure* are provided in Table 2.32. In the ASCE/SEI 7-16, target probabilities of failure in design life for Risk Categories II, III and IV are equal to 1.0%, 0.6% and 0.3%, respectively. These values are consistent with those values given in Table 4-6 in NIST (2012), which further recommends 2% probability of failure in design life for Risk Category I new *buildings,* because of their low *life safety* risk to the *occupants*. Following the FEMA P-154 approach for determining a cut-off score of 2.0 for ordinary *buildings* (i.e. Risk Category II *buildings*), the acceptable structural thresholds S_{TH} for *Very Low* to *Very High consequences of failure* are calculated by substituting the target probabilities of failure into Eq. (2.35), giving S_{TH} values of 1.7, 2.0, 2.3 and 2.6, respectively.

Table 2.32: Acceptable structural thresholds in the *Level 2 – SQST*

Risk category (ASCE/SEI 7-16)	I	II	III	IV
Consequences of failure (*Level 2 – SQST*)	Very Low	Low & Medium	High	Very High
Acceptable probability of failure	2%	1%	0.5%	0.25%
Acceptable structural thresholds, S_{TH}	1.7	2.0	2.3	2.6

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3.0 METHODOLOGY FOR SEISMIC RISK SCREENING OF EXISTING BUILDINGS: NON-STRUCTURAL SCORING SYSTEM

3.1 General

The Level 2 – Semi-Quantitative Seismic Risk Screening Tool (SQST) incorporates a non-structural scoring system, based on a simple but comprehensive seismic score that qualitatively assesses the *seismic risk* for non-structural components that are permanently attached to structures, including their supports and attachments. A global non-structural component score is determined for a *building's* most critical non-structural components. This score is compared with a specified non-structural component threshold, which corresponds to the *building's consequences of failure* for a threshold ground motion intensity (i.e., intensity VI in the Modified Mercalli Intensity scale – MMI VI). The non-structural scoring methodology focuses on non-structural hazards that pose *seismic risk* to human lives. The objective is to ensure an acceptable and consistent *seismic risk* with the focus on minimizing threats to *life safety*. The screening methodology aims to supersede the non-structural screening procedure in the 1993 NRC screening manual (NRC, 1993).

The non-structural component score corresponds to the sum of: 1) a basic seismic hazard score, based on design seismic demands prescribed by the NBC 2015, and 2) score modifiers for key parameters affecting the seismic demand and response of non-structural components, such as *Site Class*, structural response, non-structural component response, design code period, and remaining *occupancy*. The basic score considers baseline parameters for nominal response of non-structural components. The modifiers are based on prescribed and qualitative parameters that reflect the effect of amplified seismic demands on the seismic response of non-structural components rather than the probability of collapse approach employed for structural modifiers. The non-structural score is scaled to have a maximum of one hundred (100) and a minimum of zero, before accounting for the *building's* remaining *occupancy* time, where non-structural components are located. Similar to the structural scoring system, higher scores indicate lower *seismic risk* of the building. Buildings with non-structural component scores less than the specified threshold scores for different consequences of failure are flagged for component-specific seismic risk screening or seismic evaluation. Conversely, buildings with acceptable non-structural component scores are exempt from non-structural seismic evaluation.

The proposed seismic risk screening methodology considers structural and non-structural parameters used for seismic design, screening, and evaluation of non-structural components in accordance with the 2015 edition of the National *Building* Code of Canada (NBC 2015), the Canadian Standard for Seismic Risk Reduction of Operational and Functional Components (OFCs) of *Buildings*, CSA S832-14 (CSA 2014), and the NRC manual for *Seismic Risk* Screening of Non-Structural Components (NRC 1993). In addition, some parameters conform to the derived structural factors and qualitative factors based on current state of practice.

It is important to note that the purposes of the qualitative *seismic risk* screening of non-structural components in the *Level 2 – SQST* are:

1) To calculate a global non-structural component score based on *buildings'* most critical non-structural components for different vulnerabilities and consequences of failure when located in different *seismic zones* (i.e., Very Low, Low, Moderate, Moderately High, High, and Very High).

2) To exempt *buildings'* non-structural components from *Level 3 – Seismic Evaluation* when located in Very Low and Low *seismic zones*.

3) To prioritize non-compliant existing buildings for non-structural *seismic evaluation*. Non-structural components with unacceptable *seismic risk* are flagged for specific *seismic risk* screening, following the CSA S832 standard, or *Level 3 – Seismic Evaluation*.

3.2 Non-structural categories

CSA S832-14 refers to non-structural components and equipment as operational and functional components (OFCs) given the interaction between the non-structural components and the *building*'s structural system. Herein, however, the term non-structural is maintained as the *seismic risk* screening tool is based on the NBC 2015 in which non-structural components are described and categorized based on their architectural and operational function. Table 3.1 to Table 3.3 list the NBC 2015 categories for elements of structures, non-structural components and equipment, and the corresponding response sensitivity. Primary sensitivity is based on HAZUS-MH 2.1 (FEMA 2003) and ASCE 41 (ASCE 2013). The categories are divided into three major groups as follows:

1) Architectural and elements of structures: non-structural components that provide architectural functionality to the *occupants*, for example, partitions, roofing, and cladding and elements of structures such as floor and roof diaphragms that contribute to the distribution and transferring of lateral seismic forces to the main seismic force resisting system (Table 3.1);

2) Mechanical and electrical components: equipment and subcomponents that provide mechanical, electrical, plumbing, and telecommunication services to *buildings* (Table 3.2);

3) Other system components: other rigid and flexible components not included in Groups 1 and 2, Elevator equipment and rails, and pallet racks (Table 3.3).

An extensive description of different non-structural components, equipment, and *building* contents, including the sources of earthquake *damage* can be found elsewhere (FEMA E-74 2012).

Table 3.1: Architectural and elements of structures

Category	Component or equipment	Sensitivity
1	All exterior and interior walls except those in Categories 2 and 3	Drift
2	Cantilever parapet and other cantilever walls except retaining walls	Acceleration
3	Exterior and interior ornamentations and appendages	Acceleration
4	Floors and roofs acting as diaphragms	Acceleration
5	Towers, chimney, smokestacks and *penthouses*[1] when connected to or forming part of a *building*	Acceleration
6	Horizontally cantilevered floors, balconies, beams, etc.	Acceleration
7	Suspended ceilings, light fixtures[2] and other attachments to ceilings with independent vertical support	Drift
8	Masonry veneer connections	Drift
9	Access floors	Acceleration
10	Masonry or concrete fences more than 1.8 m tall[3]	Acceleration

[1] *Penthouse*s are classified as drift-sensitive in ASCE 41-13

[2] Lightning fixtures are classified as acceleration-sensitive in HAZUS-MH 2.1

[3] Cantilever elements are classified as acceleration-sensitive in HAZUS-MH 2.1

Table 3.2: Mechanical and electrical components

Category	Component or equipment	Sensitivity
11	Machinery, fixtures, equipment and tanks (including contents)	Acceleration
12	Machinery, fixtures, equipment and tanks (including contents) containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids	Acceleration
13	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a *building*	Acceleration
14	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a *building* containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids	Acceleration
15	Pipes, ducts (including contents)	Acceleration
16	Pipes, ducts (including contents) containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids	Acceleration
17	Electrical cable trays, bus ducts, conduits	Acceleration

Table 3.3: Other system components

Category	Component or equipment	Sensitivity
18	Rigid components with ductile material and connections	Acceleration
19	Rigid components with non-ductile material or connections	Acceleration
20	Flexible components with ductile material and connections	Acceleration
21	Flexible components with non-ductile material or connections	Acceleration
22	Elevators and escalators	Acceleration
23	Floor-mounted steel pallet storage racks	Acceleration
24	Floor-mounted steel pallet storage racks on which toxic or explosive materials or materials having a flash point below 38 °C are stored	Acceleration

The response sensitivity is a function of the dynamic response of the non-structural component and connections, and the *building* structure. Non-structural components with long fundamental periods and connected to flexible *building* structures typically experience amplified deformations. As a result, these components may sustain substantial *damage* due to interstorey drifts, for example, cracking and breaking of architectural components. Non-structural components with short fundamental periods and directly connected to or forming part of a structure are typically sensitive to accelerations. Lack or inadequate (non-ductile) restraint of these components may result in overturning or sliding due to amplified dynamic forces when subjected to short-period seismic demands. Some components are sensible to both drift (deformation) and acceleration, for example, exterior and interior walls.

The NBC-2015 does not categorize glazing systems. Nonetheless, it prescribes displacement limits to prevent the fall out of these elements. The displacement limits are the same as those in ASCE-7 for glass in glazed curtain walls, glazed storefronts, and glazed partitions, which are categorized as a drift-sensitive subcategory of architectural components. Following this approach for the risk scoring presented herein, glazing systems will be considered as a separate group of architectural components: architectural components forming part of a glazing system (Group 1a). Specialized contents of buildings such as furniture, fixtures, shelving, medical equipment, and art pieces are not included in the non-structural categories for seismic risk screening. Although these categories are considered by other North American standards and guidelines (FEMA 2012, CSA S832 2014), they are not explicitly categorized in the NBC 2015.

Although the non-structural component scoring system aims to determine a global score based on the most critical non-structural components, the scoring system for the three non-structural component groups, including glazing systems, is provided. This assists *building* owners to identify the risk associated with each non-structural group and to prioritize non-structural components for detailed seismic evaluation.

3.3 NBC provisions for non-structural components

3.3.1 Design periods

Since the second edition of the NBC in 1953, the seismic design of non-structural components has experienced significant changes to improve reliability and safety (Assi and McClure 2015). Determination of seismic hazard has evolved from qualitative zoning maps to probabilistic site-specific approaches. Furthermore, the NBC has changed the non-structural horizontal force factor S_p, as summarized in Table 3.4, to account for parameters that affect the component response and improve seismic demand predictions. (Note that S_p corresponds to factors and expressions in older the NBC that are equivalent to S_p in the NBC 2015.)

Table 3.4: Horizontal force factor in the NBC

NBC	Horizontal force factor, S_p
1953 to 1960[1]	$S_p = C$ C = Seismic coefficient
1965[2]	$S_p = Max(2K/R; 0.2)$ K = Seismic force coefficient, calculated as a function of *seismic zone*, type of construction, *building* importance, foundation condition, and number of storeys R = *seismic zone*
1970[3]	$S_p = C_p$ C_p = Horizontal force factor
1975 to 1985	$S_p = S_p$ S_p = Horizontal force factor
1990 and 1995	$S_p = S_p$ for architectural components $S_p = C_p A_r A_x$ for mechanical and electrical components S_p = Horizontal force factor C_p = Seismic coefficient for mechanical and electrical components A_r = Response amplification factor A_x = Height factor
2005 to 2015	$S_p = \frac{C_p A_r A_x}{R_p}$; $0.7 \leq S_p \leq 4$ S_p = Horizontal force factor C_p = Seismic coefficient for mechanical and electrical components A_r = Response amplification factor A_x = Height factor R_p = Component response factor

[1] NBC 1953 and NBC 1960 did not prescribe S_p. An equivalent S_p corresponds to the component seismic constant C for *seismic zone* 1.
[2] NBC 1965 did not prescribe S_p. An equivalent S_p corresponds to the greater of $2K/R$ and 0.2.
[3] NBC 1970 did not prescribe S_p. An equivalent S_p corresponds to the horizontal force factor C_p.

Figure 3.1 and Figure 3.2 illustrate the changes in S_p from 1953 to 2015 for typical non-structural components in *buildings*. Figure 3.1 provides S_p values for exterior and interior walls (architectural components), while Figure 3.2 provides S_p values for machinery, fixtures, equipment and tanks that are rigid and rigidly connected (mechanical and electrical components). The variation of S_p for exterior and interior walls conforms to four major periods: 1) Pre-1953, prior to the introduction of design equations, 2) 1953 to 1970, when a small S_p with a maximum value of 0.20 was prescribed, 3) 1975 to 1995, when S_p varied from 0.90 to 2; and 4) 2005 to 2015, when S_p was reduced to 0.70.

The S_p for machinery, fixtures, equipment and tanks that are rigid and rigidly connected has experienced two major changes since its introduction in the NBC 1975. The first change happened in the NBC 1980, when a significantly high *Sp* was prescribed, and the second change happened in the NBC 2005 when S_p was reduced to 80% of that in the NBC 1995. Based on the latter, two periods of time: 1975 to 1995 and 2005 to 2015 have been identified for mechanical and electrical components. Given the disproportionate value of S_p for mechanical and electrical components in the NBC 1980, the NBC 1980 was disregarded in identifying the design period. Other architectural, mechanical, and electrical components follow trends in design periods similar to those observed in Figure 3.1 and Figure 3.2.

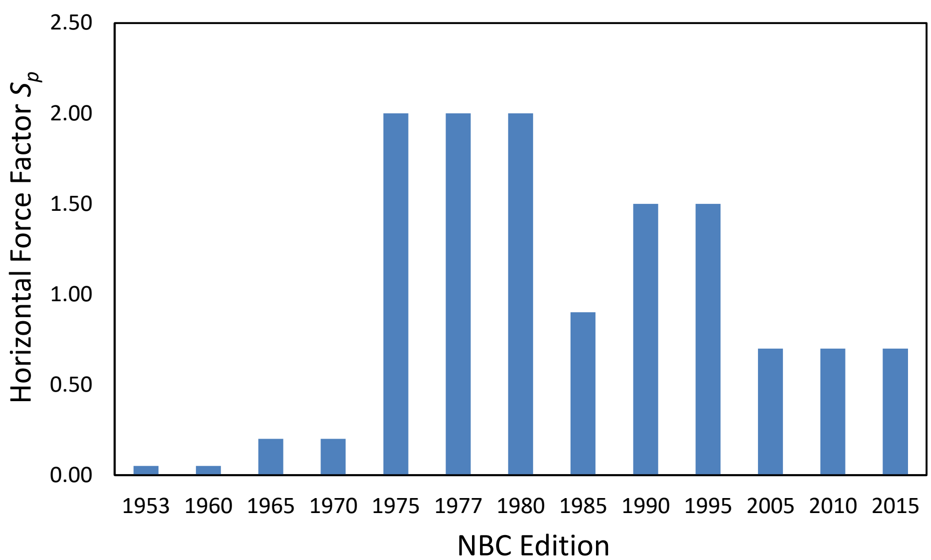

Figure 3.1: Horizontal force factor for exterior and interior walls

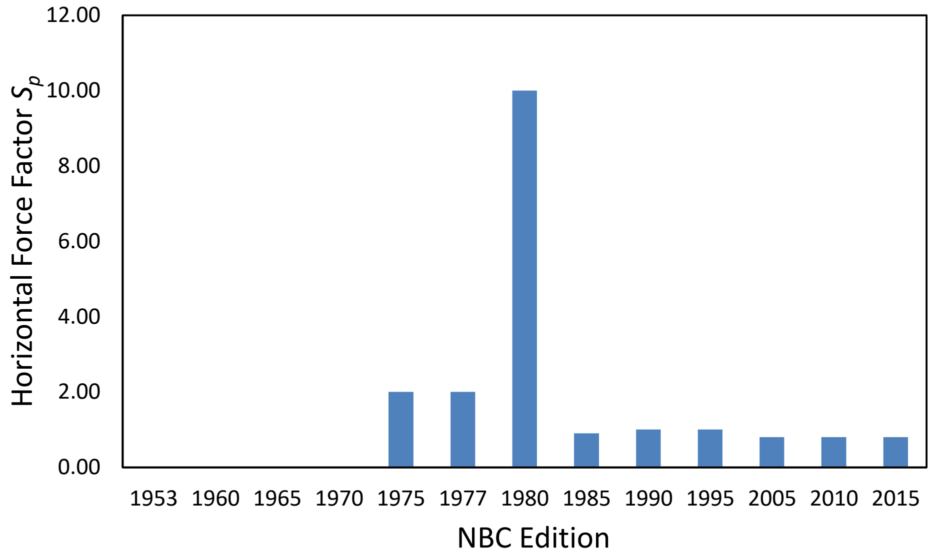

Figure 3.2: Horizontal force factor for machinery, fixtures, equipment and tanks that are rigid and rigidly connected

Based on the changes in S_p and some other factors, four design periods are identified for architectural, mechanical and electrical components:

1) Pre-1953 (NBC 1941),

2) 1953-1975 (NBC 1953 to NBC 1970),

3) 1975-2005 (NBC 1975 to NBC 1995), and

4) Post-2005 (NBC 2005 to NBC 2015)

These design periods are aligned with the changes in the component response factor R_p and corresponding elastic and inelastic design approaches. A more detailed description of the periods is provided below.

<u>Pre-1953</u>

The first edition of the NBC was published in 1941. This edition prescribed seismic design for buildings in an appendix (not in the main body of the code, and therefore not mandatory), based on the 1935 Uniform *Building* Code (UBC 1935). The NBC 1941, however, did not prescribe seismic design of non-structural components. Seismic design of architectural components was introduced in the second edition of the NBC in 1953.

<u>1953-1975 (NBC 1953 to NBC 1970):</u>

Between 1953 and 1965, the seismic forces were based on a qualitative seismic map that divided Canada into four *seismic zones*. The NBC 1953 and the NBC 1960 prescribed seismic coefficients, *C*, for four categories of architectural components. These coefficients corresponded to *seismic zone* 1 with minimum seismic hazard. For *seismic zones* 2 and 3, *C* was multiplied by 2 and 4, respectively.

In the NBC 1965, the design force for non-structural components corresponded to the maximum of $2K$ and a constant seismic coefficient of 0.2, where K was the seismic force coefficient for the *building* structure. The NBC 1965 introduced the regionalization factor R to determine seismic forces. R values of 0, 1, 2, and 4, were prescribed for *seismic zones* 0, 1, 2, and 3, respectively. In addition, the NBC 1965 incorporated the *building* importance factor I, the type of construction factor C, the foundation condition factor F, and the structural flexibility factor S.

A major change in the NBC 1970 was the introduction of the first probabilistic seismic zoning map, developed by Milne and Davenport (1969), based on expected accelerations having an annual probability of exceedance of 1% (return period of 100 years). For the design of non-structural components, the NBC 1970 prescribed an explicit formulation to design parts or portions of *buildings* and their anchorages. It introduced seven non-structural categories with their corresponding seismic coefficients, C_p, varying from 0.1 to 2, including new categories for tanks plus contents resting on the ground, and connections for exterior and interior walls. The Canadian Structural Design Manual mentioned the importance of seismic design of mechanical and electrical components for seismic demands transferred from the *building* structure, without providing specific design equations. This was not in the main body of the code and therefore was not mandatory. The manual also recommended seismic displacement requirements based on an interstorey drift limit of 0.5% and inelastic deflections of the *building* structure corresponding to three times the calculated elastic deflections.

<u>1975 to 2005 (NBC 1975 to NBC 1995)</u>

In the NBC 1975, a new design equation was introduced to determine the seismic force of non-structural components, based on the horizontal force factor S_p. This equation, however, did not consider importance, type of construction, foundation condition, and structural flexibility of the *building* structure. The seismic hazard was based on the zoning map developed for the NBC 1970. The NBC 1975 was the first edition of the NBC that prescribed requirements for seismic design of mechanical and electrical components. Machinery, fixtures and equipment, pipes, and tanks plus contents were considered in the same category as towers, chimneys, smokestacks and *penthouse*s, and were prescribed with an S_p of 2. In general, S_p values for architectural components were greater than the seismic coefficients prescribed in previous editions of the NBC, for example, for exterior and interior walls, S_p in the NBC 1975 was approximately 10 times greater than the C_p (equivalent to S_p) prescribed in the NBC 1970 (Figure 3.1).

Between 1977 and 1990, the seismic force equation for non-structural components did not experience a significant change. In the NBC 1977, the seismic zoning maps and seismic provisions remained essentially the same as those in the NBC 1970. In the NBC 1985, however, new seismic maps, based on a lower probability of exceedance of 10% in 50 years (return period of 475 years), were introduced. The NBC 1985 divided the category for mechanical and electrical components in two subcategories to separate rigid and rigidly connected components from other components (i.e., flexible components). The NBC 1985 prescribed S_p of 0.9 for rigid mechanical and electrical components and S_p of 4.4 for other components. These values of S_p were substantially lower than the values in the NBC 1980. Furthermore, machinery, fixtures and equipment, pipes and tanks plus contents were listed in a separate category with an S_p of 10, which was significantly higher than that prescribed in other editions of the NBC (Figure 3.2).

In the NBC 1990, an equation was introduced to determine S_p for mechanical and electrical components. The parameters in this equation are the seismic coefficient factor C_p, the amplification factor A_r, and the height factor A_x. C_p accounts for the component's relative importance and the content of *hazardous materials*, which varies from 0.7 to 1.5. A_r represents the component's dynamic amplification, based on the component's dynamic response relative to the *building* structure; A_r is equal to 1.0, 2.0, and 4.5, for rigid, flexible, and other components, respectively. A_x represents the acceleration amplification as a function of the *building* elevation; A_x varies linearly from 1.0 at ground to 2.0 at roof level. The NBC 1990 is, therefore, the first edition of the NBC that considers the importance of components and the effect of *hazardous materials* and the dynamic amplification of the response due to the non-structural component's characteristics and the *building* structure. The values of S_p in the NBC 1990 varied from 0.7 to 6.5 and were, in general, greater than those in the NBC 1985. The seismic forces in the NBC 1990 were based on seismic hazard values incorporated in the NBC 1985. More stringent requirements were prescribed for the seismic displacement of non-structural components. Displacement limits corresponding to *building* interstorey drift ratios of 1% and 2% were specified for post-disaster and all other *buildings*, respectively. Furthermore, the NBC 1990 prescribed structural response factors *R,* varying from 1.0 to 4.0, to determine the expected inelastic deformations of the *building* structure, rather than the fixed value of 3.0 prescribed in previous NBC editions.

In the NBC 1995, the equations to determine design seismic forces were similar to those in the NBC 1990, with the exception of the importance factor *I,* which was reintroduced to account for increased risks for post-disaster and school *buildings*. (The importance factor had been removed from the seismic force equations since the NBC 1970.) The A_r values were reduced for components located on ground level that are flexible or flexibly connected, for non-brittle pipes and ducts, as well as for all other components. The NBC 1995 used the same seismic hazard maps as the NBC 1990. No changes in seismic displacement requirements for non-structural components were incorporated in the NBC 1995.

Post-2005 (NBC 2005 to NBC 2015)

The NBC 2005 adopted site-specific Uniform Hazard Spectra (UHS) with probability of exceedance of 2% in 50 years and a corresponding return period of 2475 years for seismic design of structural and non-structural components. The UHS' lower probability provides a more uniform margin of collapse, closer to the structural failure probability (Heidebrecht 2003). The NBC 2005 introduced the current general equation to determine the seismic demand of all types of non-structural components, including architectural components. This equation is based on peak ground acceleration, determined as 0.3 times the product of the spectral acceleration at 0.2 seconds and corresponding acceleration-based site coefficient F_a. The design equation assumes that non-structural components are controlled by acceleration at short periods.

The NBC 2005 increased the component amplification A_r to a maximum of 2.5 and the maximum floor amplification A_x to 3, and introduced the component response factor R_p. By introducing R_p, the NBC recognized the energy dissipation capacity of non-structural components and connections. As a result, the seismic demand for non-structural components was reduced. While the reduction in demand was significant for architectural components (Figure 3.1), it was minimal for mechanical and electrical components (Figure 3.2).

The changes in the NBC 2005 for the seismic design of non-structural components followed the modifications in seismic force provisions in the 1994 NEHRP (NEHRP 1995) and the 1997 NEHRP (FEMA 303 1997). Relative to the previous edition, the NEHRP 1994 increased the maximum component amplification factor a_p from 2 to 2.5, based on recommended revisions by Soong et al. (1993). The 1994 NEHRP reintroduced the height factor $(1 + z/h)$ to account for the linear distribution of floor acceleration amplification over the *building's* height. The height factor was present in the 1985 and 1988 editions of the NEHRP (NEHRP 1985 and 1988), but it was removed for the 1991 edition (NEHRP 1991). In addition, the NEHRP introduced the response modification factor R_p and the component importance factor I_p, to consider the ductility of connections and the consequences of failure, respectively. In the 1997 edition of the NEHRP, the height factor was modified to $(1 + 2\,z/h)$ to increase floor acceleration amplification to a maximum of 3. The redefined height factors provide a reasonable bound to floor acceleration amplification to large Californian earthquakes, specifically to earthquake motions with peak ground accelerations greater than 0.1g. (Drake and Bachman [1995 and 1996] had previously reported maximum amplifications of 4.)

The seismic force and displacement requirements, including the seismic hazard, for non-structural components did not significantly change from 2005 and 2015. In the NBC 2015, elevators and escalators (Category 22), as well as pallet storage racks (Categories 23 and 24), were introduced in the list of elements of structures and non-structural components and equipment. In addition, the NBC 2015 introduced specific displacement limits for glazed architectural components, including glazed curtain walls, glazed storefronts, and glazed partitions.

3.3.2 **Current equation for seismic design**

The seismic design equation in the NBC 2015 is the basis of the seismic risk scoring for non-structural components. Design force V_p is determined as follows:

$$V_p = 0.3F_aS_a(0.2)I_ES_pW_p \tag{3.1}$$

where F_a is the site coefficient, $S_a(0.2)$ is the spectral acceleration at the short period of 0.2 seconds, I_E is the building importance factor, S_p is the non-structural horizontal force factor, and W_p is the seismic weight of the non-structural component. S_p is determined as follows:

$$S_p = C_pA_rA_x/R_p; \quad 0.7 \le S_p \le 4 \tag{3.2}$$

where C_p is the component factor accounting for the risk associated with component failure, A_r is the force amplification factor representing the dynamic amplification of the component as a function of the ratio of the component's natural period to the *building* structure's fundamental period, A_x is the height factor $(1 + 2\,h_x/h_n)$ representing the amplification of the acceleration from the base of the *building* structure to the height at which the component is attached, and R_p is the component response modification factor considering the component's energy dissipation capability and its connection to the structure.

A similar equation for seismic design is prescribed by North American standards ASCE 7-16 (ASCE 2016) and ASCE-41 (ASCE 2017). This equation is as follows:

$$F_p = \frac{0.4a_pS_{DS}\left(1 + 2\frac{z}{h}\right)}{\left(\frac{R_p}{I_p}\right)}W_p \tag{3.3}$$

$$F_p(minimum) = 0.3S_{DS}I_pW_p \tag{3.4}$$

$$F_p(maximum) = 1.6S_{DS}I_pW_p \tag{3.5}$$

where, F_p is the seismic design force for the non-structural component, a_p is the component amplification factor, S_{DS} (S_{XS} in ASCE 41-17) is the spectral acceleration response at short periods, W_p is the operating weight of the component, $(1 + 2\,z/h)$ is the location of the non-structural component factor (z [x in ASCE 41-17] corresponds to the elevation on the structure of the component's point of attachment with respect to the base, while h corresponds to the elevation of the *building's* roof, respectively), R_p is the response modification factor, and I_p is the component importance factor.

Table 3.5 provides a comparison of the parameters included in the seismic design equations for non-structural components in the NBC 2015, ASCE 7-16, and ASCE 41-17 provisions. The *seismicity* (peak ground acceleration) in ASCE 7-16 and ASCE-41-17, expressed as $0.4S_{DS}$, is equivalent to the term $0.3F_aS_a(0.2)$ in the NBC-2015. S_{DS} corresponds to two thirds of the risk-targeted maximum considered earthquake acceleration, S_{MS}, which is comparable to the design spectral response acceleration plateau at short periods of the design earthquake in the NBC 2015. Structural commentary J in the NBC 2015 (NBC 2015) states that $0.3I_EF_aS_a(0.2)$ is equivalent to the expected peak acceleration at the base of the *building* (based on experience) and is approximately equal to the seismic hazard expressions in FEMA 368 (FEMA 2000), which are the same as those in 2000 NEHRP. The comparison in Table 3.5 separates the importance factor I_E from the seismic hazard term since it accounts for consequences of failure based on importance category. Given this, the consequences of failure in the NBC 2015 corresponds to the product of I_E and component factor C_p. ASCE 7-16 and ASCE 41-17 implement the component importance factor I_p to account for consequence of failure. This factor may be considered as equivalent to C_p in the NBC 2015. The factor capturing the amplification in components' seismic response, which consists of amplification, height, and component response factors, is essentially the same for the three provisions.

Table 3.5: Parameters in seismic design equation for non-structural components

Code	Seismic hazard	Consequence	Component response
NBC 2015	$0.3F_aS_a(0.2)$	I_EC_p	$S_p = \frac{A_rA_x}{R_p}$ $A_x = 1 + 2\frac{h_x}{h_n}$
ASCE 7-16 and ASCE 41-17	$0.4S_{DS}$ $S_{DS} = \frac{2}{3}F_aS_{MS}$	I_p	$\frac{a_p(1 + 2\,z/h)}{R_p}$

3.4 *Level 2 – SQST* methodology: Non-structural components

3.4.1 Non-structural seismic risk scoring

The non-structural component *seismic risk* score consists of a quantitative basic score that reflects the seismic demand of reference non-structural components and qualitative modifiers that account for the change in seismic demand due to the variation of structural and non-structural parameters. Given this, the overall non-structural component score is qualitative in nature. The non-structural component score is based on the performance and vulnerability of non-structural components and the potential consequences to life safety rather than the probability of collapse approach followed for the structural *seismic risk* scoring. To simplify and facilitate the non-structural component screening, the non-structural component scores are calculated globally for the building accommodating different groups of non-structural components, assuming the most critical components as the worst case scenario. In other words, the non-structural component score is globally assigned to the building, rather than to a specific non-structural component. Nevertheless, the tool has the capacity to calculate the score for specific groups of non-structural components.

The non-structural component score, *NS*, is based on the non-structural index, *NI*, which incorporates the main structural and non-structural parameters affecting the non-structural component response. Similar to the risk index *RI* in CSA S832-14, the *NI* index is expressed as the product of non-structural vulnerability indexes and the consequences of failure, as follows:

$$NI = NV_B \times \prod_i NV_i \times C \tag{3.6}$$

where NV_B is the basic non-structural vulnerability index, NV_i is the non-structural vulnerability index corresponding to non-structural and structural parameters, and C is the consequences of failure factor.

Acceptable *NI* should be less than or equal to the non-structural risk threshold NR_{TH} as follows:

$$NI \leq NR_{TH} \tag{3.7}$$

The non-structural component score, *NS*, expressed in a summation form, results from multiplying both sides of Eq. (3.6) by the negative common logarithm, as follows:

$$NS = -\log_{10}(NI) = NS_B + \sum_i NM_i \tag{3.8}$$

where NS_B is the non-structural component basic score, calculated from the seismic coefficient of a reference non-structural component, and NM_i are non-structural component score modifiers corresponding to structural and non-structural parameters. NM_i values are determined by varying structural and non-structural behavioural parameters and comparing the resulting *NS* with NS_B.

Combining C and NR_{TH} and then applying the negative common logarithm results in the non-structural component score thresholds, NS_{TH}. Acceptable *NS* should be equal to or greater than NS_{TH}. (Note that applying the negative logarithm results in a different inequality.)

$$NS \leq NS_{TH} \tag{3.9}$$

3.4.2 Seismic risk screening methodology

Figure 3.3 illustrates the methodology for scoring the *seismic risk* for existing buildings due to non-structural components. Non-structural component scoring is performed in four steps, as follows:

1) Determining the basic non-structural component score NS_B, based on the 5% damped spectral response acceleration value at 0.2 s, $S_a(0.2)$, for the location of the building where the non-structural components are mounted in accordance with the NBC 2015.

2) Determining non-structural component score modifiers NM_i for different parameters. (The subscript "*i*" refers to the modifying parameter.) Five non-structural component score modifiers are determined:

    a. Site Class modifier, NM_{SC},

    b. structural response modifier, NM_{SR},

c. component response modifier, NM_{CR},

d. design period modifier, NM_{DP}, and

e. remaining occupancy time modifier, NM_{RO}.

3) Calculating the non-structural component score NS, by adding NS_B to the sum of NM_i.

4) Comparing NS with the non-structural component score threshold NS_{TH}, to determine whether the seismic risk is acceptable ($N_S \geq NS_{TH}$). If $N_S < NS_{TH}$, seismic risk screening or seismic evaluation of specific components may be required.

Further details of the scoring system are provided below.

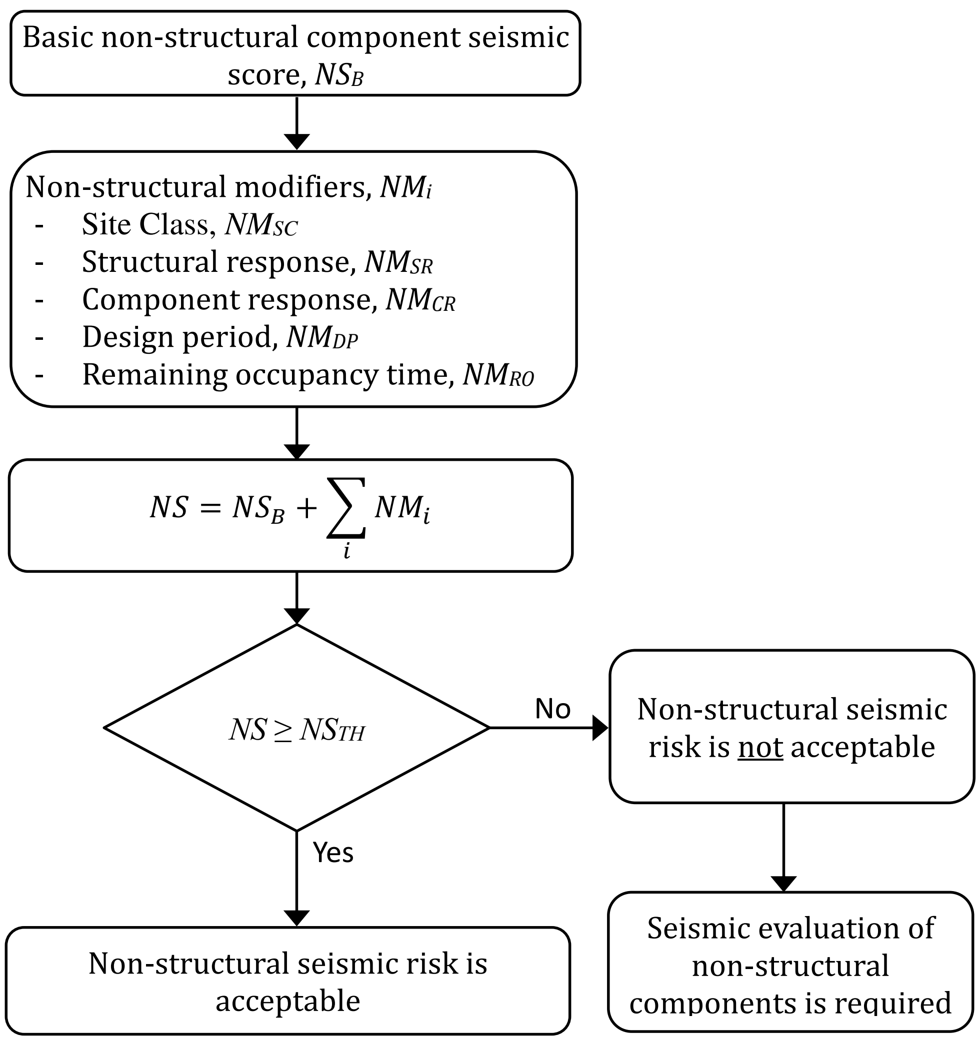

Figure 3.3: *Seismic* risk assessment of non-structural components

3.5 Basic non-structural score

The non-structural score NS_B is calculated for different seismic coefficients – $C_{Sp} = V_p/W_p$, which are obtained from the design seismic force equation in the NBC 2015, as follows:

$$C_{Sp} = V_p/W_p = 0.3F_aS_a(0.2)I_ES_p \tag{3.10}$$

N_{SB} is a function of the 5% damped spectral response acceleration at 0.2 seconds, $S_a(0.2)$, for a reference non-structural component with nominal horizontal force factor (S_p = 1.0), in a low rise *building* (i.e., less than three storeys with seismic response controlled by short periods) located on *Site Class* C (*F(0.2)* = 1.0). The nominal S_p assumes rigid components (A_r = 1.0) that are located on the ground floor (A_x = 1.0) and have nominal ductility capacity (R_p = 1.0). The C_{Sp} coefficient for NS_B excludes the importance factor I_E and component factor C_p. These factors increase the seismic demand for expected level of performance and consequences of failure for non-structural components. Given this, I_E and C_p are used to establish the acceptable non-structural score thresholds.

The 5% damped spectral response acceleration at the short period of 0.2 second, $S_a(0.2)$, corresponds to an earthquake scenario with 2% probability of occurrence in 50 years. Short-period 5% damped spectral response accelerations predominantly amplify the response of rigid non-structural components that are sensitive to accelerations, and in general, provide a critical design force for all types of components, including those that are sensitive to drifts. $S_a(0.2)$ values are selected from the seismic design data for selected locations in Canada, as provided in Appendix C of Division B of the NBC 2015.

NS_B scores are determined as a function of the negative logarithm of the seismic coefficient, C_{Sp}, as follows:

$$NS_B = 26\left[-log_{10}\left(V_p/W_p\right) + 1.26\right] \tag{3.11}$$

The coefficients 26 and 1.26 in Eq. (3.11) are introduced to avoid negative values of NS_B and to obtain a maximum non-structural score of 100 before considering a bonus score for remaining *occupancy* time.

Table 3.6 provides NS_B calculated with Eq. (3.11) for C_{Sp} values corresponding to $S_a(0.2)$ for different Canadian *seismic zones*, as defined in Table 2.8. NS_B for other $S_a(0.2)$ can be interpolated from the data in Table 3.6. NS_B decreases with the increase in *seismicity*, reflecting the detrimental effect of earthquake ground motions on the *seismic risk* of non-structural components. NS_B is applicable to non-structural components located in any model *building type*.

Table 3.6: Basic non-structural scores

Seismic zone	$S_a(0.2)$	V_p/W_p	NS_B
VL	0.052	0.02	80
Sa(0.2) ≤ 0.10	0.07	0.02	76
	0.085	0.03	74
	0.1	0.03	72
L	0.13	0.04	69
0.10 < Sa(0.2) ≤ 0.20	0.15	0.05	68
	0.18	0.05	65
	0.2	0.06	64
M	0.24	0.07	62
0.20 < Sa(0.2) ≤ 0.35	0.28	0.08	60
	0.32	0.10	59
	0.35	0.11	58
MH	0.45	0.14	55
0.35 < Sa(0.2) ≤ 0.75	0.55	0.17	53
	0.65	0.20	51
	0.75	0.23	49
H	0.85	0.26	48
0.75 < Sa(0.2) ≤ 1.15	0.95	0.29	47
	1.05	0.32	46
	1.15	0.35	45
VH	1.3	0.39	43
1.15 < Sa(0.2) ≤ 1.73	1.45	0.44	42
	1.6	0.48	41
	1.73	0.52	40

3.6 Non-structural component score modifiers

Non-structural component modifiers NM_i assess the global effect of different parameters on the *seismic risk* of non-structural components. The non-structural component modifiers identified in Figure 3.3 are determined by varying the parameter values affecting the non-structural components' seismic response. Non-structural component modifiers are obtained by comparing the modified non-structural score and basic non-structural score as follows:

$$\begin{aligned} NM_i = NS_M - NS_B \\ &= 26\lfloor -log_{10}(C_{Sp}|condition\ i) + 1.26\rfloor \\ &- 26\lfloor -log_{10}(C_{Sp}) + 1.26\rfloor \end{aligned} \tag{3.12}$$

Eq. (3.12) is simplified to determine NM_i based on the negative common logarithm of the factor, F_i, affecting the seismic demand, as follows:

$$NM_i = 26[-log_{10}(F_i)] \quad (3.13)$$

The resulting NM_i are rounded to near integers to simplify the *Level 2 – SQST* and to facilitate *seismic risk* screening. Positive and negative values of NM_i represent score credits and penalties, respectively, corresponding to the beneficial and detrimental effects that the modifying parameters have on the *seismic risk* of non-structural components. The total non-structural modifier, NM_T, results from summing the individual modifier for each non-structural and structural parameter.

In addition to global score modifiers, the *Level 2 – SQST* methodology defines non-structural modifiers that permit the *seismic risk* screening of groups of non-structural components. This provides additional information that facilitates risk mitigation planning based on critical non-structural components.

3.6.1 **Site Class modifier**

The characteristics of the *building* site, specifically the stiffness and wave propagation velocity, affects the amplitude of the seismic demand. As compared to hard rock, soft soils with low wave propagation velocity amplify ground vibrations, increasing the *building* structure's fundamental periods due to the soil-structure interaction effect. This may cause resonance between the supporting soil and high-rise *buildings* with long fundamental periods. Conversely, short-period (higher frequency) vibrations transferred by very stiff soils significantly affect the response of low-rise *buildings* governed by short fundamental periods. Given the interaction with the *building* structure, non-structural components are also affected by the site's seismic response. Flexible components with long fundamental periods are controlled by amplified displacements and lateral drifts, while stiff components with short fundamental periods are controlled by amplified forces. To capture the effect of the soil on the seismic response of *building* structures and non-structural components, the NBC 2015 specifies site coefficients for spectral response acceleration for different *Site Classes* and different reference peak ground acceleration (PGA_{ref}) values. The site coefficients increase with the increase in soil softness (decrease in wave propagation velocity), the increase in the *building's* fundamental period, and the decrease in PGA_{ref}.

Table 3.7 provides prescribed site coefficients in the NBC 2015 for the short fundamental period of 0.2 seconds, which range from 0.69 for *Site Class* A (hard rock) to 1.64 for *Site Class* E (soft soil). The use of site coefficients at 0.2 seconds assumes seismic response of non-structural components controlled by accelerations rather than drifts. Note that Table 3.7 does not include *F*(0.2) values for *Site Class* A with average shear wave velocity greater than 1500 m/s. These values are ignored in the *Level 2 – SQST*. Also note that site-specific evaluation is required to determine *F*(0.2) for *Site Class F*, which corresponds to other soils such as liquefiable soils, quick and highly sensitive clays, and severe geological hazards that have the potential to significantly degrade seismic performance.

Table 3.7: Site coefficients for short fundamental period of 0.2 s (the NBC 2015)

Site Class	***F*(0.2)**				
	$PGA_{ref} \leq 0.1$	**$PGA_{ref} = 0.2$**	**$PGA_{ref} = 0.3$**	**$PGA_{ref} = 0.4$**	**$PGA_{ref} \geq 0.5$**
Class A (Hard rock)	0.69[1]	0.69[1]	0.69[1]	0.69[1]	0.69[1]
Class B (Rock)	0.77	0.77	0.77	0.77	0.77
Class C (Very dense soil/soft rock)	1.00	1.00	1.00	1.00	1.00
Class D (Stiff Soil)	1.24	1.09	1.00	0.94	0.9
Class E (Soft Soil)	1.64	1.24	1.05	0.93	0.85
Class F (Other Soils)	2	2	2	2	2

[1] For shear wave velocities, $\bar{V}_{s30}$, measured in-situ, the *F(0.2)* values for *Site Class* A are permitted to be multiplied by the factor $0.04 + (1500/\bar{V}_{s30})$.
[2] Site-specific evaluation is required to determine *F*(0.2).

Table 3.8 provides the *Site Class* modifiers NM_{SC} calculated with Eq. (3.13). Minimum and maximum NM_{SC} of -5 and 5 are determined for *Site Class* E and A, respectively. Note that NM_{SC} for *Site Class* A corresponds to *F*(0.2) of 0.69. Permitted lower values of *F*(0.2) for *Site Class* A, based on in-situ measurements, are not considered for NM_{SC}. Similar to *building* structures, non-structural components in *buildings* located on *Site Class F* are flagged for *Level 3 – Seismic Evaluation,* given the required site-specific evaluation of site condition.

Table 3.8: Non-structural *Site Class* modifier

Soil Class	NM_{SC}				
	PGA_{ref} <= 0.1	**PGA_{ref} = 0.2**	**PGA_{ref} = 0.3**	**PGA_{ref} = 0.4**	**PGA_{ref} >= 0.5**
Class A (Hard rock)	5	5	5	5	5
Class B (Rock)	3	3	3	3	3
Class C (Very dense soil and soft rock)	0	0	0	0	0
Class D (Stiff soil)	-2	-1	0	1	1
Class E (Soft soil)	-5	-2	-1	1	2
Class F[1] (Other soils)	-	-	-	-	-

[1] *Site Class F* triggers *Level 3 – Seismic Evaluation* of non-structural components based on site-specific evaluation.

3.6.2 Structural response modifier

The *seismic risk* of non-structural components highly depends on the building structure's response. In general, non-structural components' *seismic risk* increases with the onset of structural deficiencies due to *irregularities*, *pounding*, as well as *deterioration* and *age*. Structural deficiencies may significantly amplify floor accelerations, which in turn amplify the seismic demands on non-structural components. Components located on weak or soft *storeys*, irregular plans or at the edges of *buildings*, far from the centre of resistance, are subject to significant amplification of seismic demands. *Deterioration* and *age* may compromise the structural strength and drift capacity of the *building*, increasing the *building's* probability of collapse (Aldeka 2015).

Despite the building response effect, current design provisions, including North American NBC 2015, ASCE 7-16, and ASCE 41-2017, New Zealand NZS 4219 (NZS 4219 2009) and European Eurocode 8 (CEN 2004), do not explicitly consider structural characteristics and deficiencies for seismic design of non-structural components. These provisions focus solely on the amplification of seismic forces due to the characteristics of non-structural components, based on horizontal force factor S_p and floor amplification at different building heights. The 1993 NRC screening manual and the Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete *Buildings* (JBDPA 2001), however, quantify the amplification of non-structural components' seismic forces due to *building* deficiencies. In addition, the second edition of FEMA 154 (2002) provides a quantitative methodology to determine score modifiers for *irregularities* and pre-code design, which apply to the seismic screening of existing *buildings*. Similarly, Tischer et al. (2014) proposed increases in demand of up to 3.5 times for severe *building* weaknesses, including *irregularities*, *pounding*, and *deterioration* for an adapted rapid seismic screening method for evaluating school *buildings*.

The structural response effect is considered with the structural response modifier NM_{SR}, which is based on the increase in non-structural components' seismic demand. NM_{SR} reflects expected higher seismic demands and the associated increases in seismic vulnerability of non-structural components due to the following structural conditions:

- *Building type* (*BT*)
- *Building* vertical irregularity (*VI*)
- *Building* horizontal irregularity (*HI*)
- *Pounding* (*PO*)
- *Deterioration* and *age* (*DA)*

The NM_{SR} factor corresponds to the sum of the contributions of the corresponding modifiers for the structural parameters affecting the response, as follows:

$$NM_{SR} = NM_{BT} + NM_{VI} + NM_{HI} + NM_{PO} + NM_{DA} \tag{3.14}$$

where NM_{BT}, NM_{VI}, NM_{HI}, NM_{PO} and NM_{DA} are the non-structural modifiers for *building type*, vertical *irregularity*, horizontal *irregularity*, *pounding*, and *deterioration* and *age*, respectively.

NM_{SR} has a minimum value of -20 to account for a maximum expected amplification of seismic demand of 5, rather than the combined contribution to increasing seismic demand by the individual structural modifiers, which would result in an unreasonably high amplification factor of approximately 30. As illustrated in Figure 3.4, the maximum floor acceleration amplification of 5 provides a reasonable upper bound that covers almost the totality of more than 2200 recorded floor accelerations for earthquakes in California that occurred between 1978 and 2010 (Fathali and Lizundia 2011). The data in Figure 3.4 are from *buildings* with different structural conditions, including deficiencies and redundancies. Similar data were used to determine the limits for the height factor and the horizontal seismic force coefficient in the 1994 and 1997 editions of the NEHRP (FEMA 303 1997; Drake and Bachman 1995, 1996). These limits have been incorporated in the UBC 1997, ASCE 7 7-16, and the NBC 2015. For *buildings* in Eastern and Western Canada, Saatcioglu et al. (2008) predicted floor acceleration amplifications of approximately 4 for five-*storey* frame *buildings* and proposed an equation that results in maximum floor amplifications of 5 for *buildings* with short periods. Limitations in seismic demand for non-structural scoring were acknowledged in the 1993 NRC screening manual by considering amplification factors ranging from 2 to 4 for different building types and irregularities.

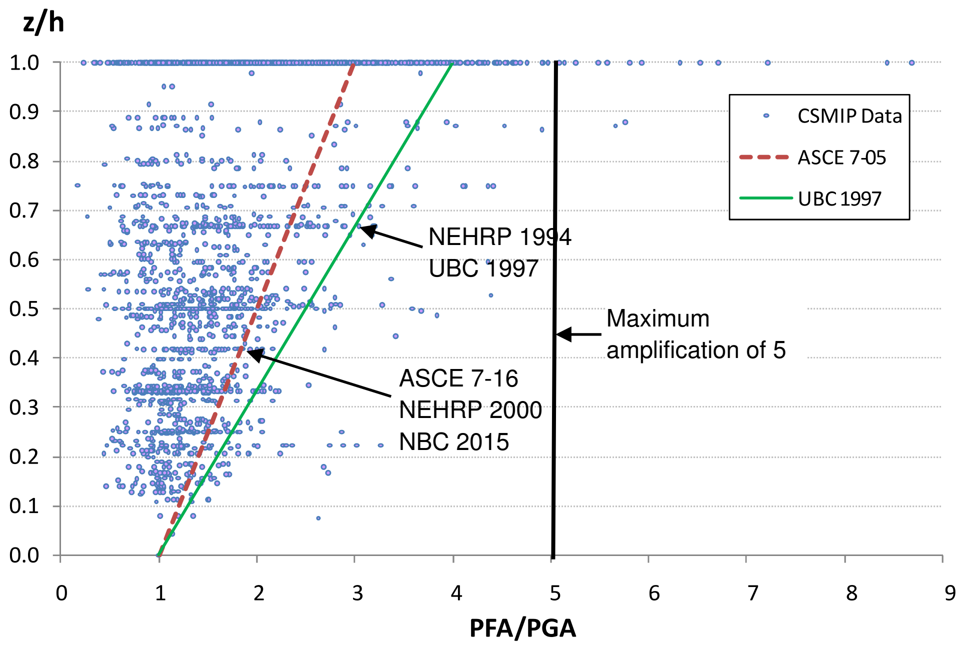

Figure 3.4: Amplification of seismic demand, expressed as the ratio of peak floor acceleration to peak ground acceleration (PFA/PGA), for floor height to roof ratio (*z/h*) (modified from Fathali and Lizundia 2011)

3.6.2.1 <u>Building type</u>

The *building type* modifier NM_{BT} estimates the effect of the seismic force-resisting system's (SFRS) flexibility on the amplification of seismic demands that are transferred to non-structural components. NM_{BT} is based on the displacement response of *buildings* with different heights: low-rise (one to three storeys), mid-rise (four to seven storeys), and high-rise buildings (more than seven storeys). (The building height classification corresponds to that defined for seismic risk screening of building structures, which is consistent with the height classification in FEMA P-154.)

<u>Amplification of seismic demand</u>

The amplification factor is obtained by comparing the expected peak displacement u_{max} for relatively flexible building types with that of relatively stiff building types. The 1993 NRC screening manual recognized the effect of building flexibility by considering irregularity due to relatively flexible steel and concrete moment frame *building types* (SMF and CMS, respectively). Following this approach, the non-structural component scoring system considers two building type amplification factors for two corresponding groups of *building* SFRSes, as follows:

1) Group 1: relatively flexible wood light frame (WLF), wood post and beam (WPB), steel moment frame (SMF), steel light frame (SLF), concrete moment frame (CMF), and precast concrete frame (PCF) buildings.

2) Group 2: relatively stiff braced steel frame (SBF), steel frame with masonry infill walls (SIW), concrete frame with masonry infill walls (CIW), concrete shear walls (CSW), pre-cast concrete shear walls (PCW), reinforced masonry bearing-wall buildings (RML and RMC), unreinforced masonry bearing-wall buildings (URM), and manufactured homes (MH).

The values of u_{max} are determined with Eq. 2.28, using response spectrum analysis (Chopra 2007) for acceleration- and velocity-controlled responses, as defined in Section 2.5.8 for the structural *pounding* modifier. Table 3.9 provides the values of u_{max} for the two groups of SFRSes for different *seismic zones* and *building* heights. The u_{max} for each group corresponds to the average of the u_{max} values of the *building type*s contained in the group. Amplification factors varied for low-rise *buildings*, while they remained unchanged for mid-rise and high-rise *buildings*. The invariability of amplification factors for mid-rise and high-rise *buildings* results from u_{max} ratios that are generally independent of *seismicity*. For long natural periods *T*, the effect of *seismicity* is canceled due to u_{max} for both groups of *buildings* being controlled by the same long-period design spectral response acceleration. The global amplification factor for all *seismic zones* is approximately 2 for low-rise *buildings* and approximately 1.3 for mid-rise and high-rise *buildings*.

Table 3.9: Amplification of elastic demands due to building type for different building heights

Seismic zone	Amplification factor		
	Low-rise	Mid-rise	High-rise
VL	1.83	1.34	1.28
L	2.06	1.34	1.28
M	1.95	1.34	1.28
MH	1.70	1.34	1.28
H	1.98	1.34	1.28
VH	2.06	1.34	1.28
Average	1.93 ≈ 2	1.34 ≈ 1.3	1.28 ≈ 1.3

Determination of NM_{BT}

NM_{BT} is determined with Eq. (3.13) as a function of the amplification of elastic demand factors of 2 for low-rise *buildings* and 1.3 for mid-rise and high-rise *buildings*. The corresponding global NM_{BT} are -8, and -3, respectively. These general modifiers are applicable to global, and group-specific *seismic risk* screening.

3.6.2.2 Building vertical and horizontal irregularity modifiers

The *building* vertical and horizontal *irregularity* modifiers NM_{VI} and NM_{HI} are determined for the expected amplification of seismic demand due to *irregularities* of *buildings* containing non-structural components, based on prescribed factors and principles of energy dissipation.

Building irregularity factor and amplification of seismic demand

NM_{VI} and NM_{HI} are based on a comparison of amplification factors in existing seismic screening methodologies and calculated amplification factors corresponding to reduced maximum drift capacity in accordance with the scoring methodology in FEMA P-154 (Third edition; FEMA 2015). The existing methodologies include the 1993 NRC screening manual (NRC 1993), FEMA 154 (Second edition; FEMA 2002), and the Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (Japanese Standard for Seismic Evaluation; JBDPA 2001).

The 1993 NRC screening manual prescribes factors of 1.3, 1.5, and 2 for vertical, horizontal, and soft-storey *irregularities*, respectively, to determine the structural seismic index. Among these *irregularities*, soft *storey* and torsion correspond to major *building irregularities* that affect the seismic response of non-structural components. The 1993 NRC screening manual determines the non-structural index based on a single factor that considers the presence of at least one undesirable condition, without discriminating the type of *irregularity*. It also increases the non-structural index for pre-1970s and post-1970s life-safety components by factors of 2 and 1.5, respectively, and for vital-operation components by a factor of 2. The lower factor of 1.5 reflects the expected better *building* performance due to improved seismic design provisions introduced in the NBC 1970 and subsequent editions. Disregarding *building* design year, which is considered separately in the *Level 2 – SQST*, the basic *irregularity* factor for all types of non-structural components in the 1993 NRC screening manual is 2.

The second edition of FEMA 154 (2002) prescribes score modifiers for vertical *irregularity* based on engineering judgement. Score modifiers are determined such that *buildings* with vertical *irregularity* in high and moderate *seismicity* regions are flagged for detailed evaluation. For *buildings* in low *seismicity* regions, the score modifiers are selected to ensure that the sole application of the modifier does not result in scores lower than the acceptable threshold. The horizontal *irregularity* score modifier is calculated by increasing seismic demand by 50% (i.e., amplification factor of 1.5).

The third edition of FEMA P-154/P-155 (2015) follows the OSHPD HAZUS procedure to determine *irregularity* modifiers based on probability of collapse. For Level 1 scoring, FEMA P-154/P-155 addresses the effect of *building irregularities* by modifying the parameters used to develop the capacity and fragility curves, as well as collapse rates rather than modifying seismic demand. Vertical and horizontal *irregularity* modifiers are calculated by reducing the median value of interstorey drift ratio corresponding to complete structural *damage* state Δ_C by factors of 1.25 and 2 for vertical and horizontal *irregularities*, respectively.

Although the approach to calculating score modifiers for *building irregularities* changed in the third edition of FEMA P-154/P-155 from amplifying seismic demand to reducing interstorey drift ratios, amplification factors for seismic demands can be determined by comparing the inelastic demand of an irregular *building* with that of the *model building type*. Inelastic demand results from reducing elastic demand by the factors of $R=1/\mu$ and $R=1/(2\mu-1)^{1/2}$ for constant velocity (long periods) and constant acceleration (short periods) regions of the spectra, respectively, based on the rules of equal displacement and equal energy (Newmark and Hall 1982). Given the displacement ductility definition as the ratio of Δ_C to yield displacement, $\mu = \Delta_C/\Delta_y$, the reductions in μ are proportional to the reductions in Δ_C. Following this, the amplification in elastic demands due to vertical and horizontal *irregularities* are calculated for μ ranging from 2 to 8, as reported in Table 3.10 and Table 3.11. For *buildings* with vertical *irregularities*, the average increases in elastic demand are 1.25 and approximately 1.15 for long-periods and short-periods, respectively, while for *buildings* with horizontal *irregularities,* the values are 2 and approximately 1.5 for long-periods and short-periods, respectively. The factor of 1.5 for short periods for horizontal *irregularities* is consistent with the amplification factor used in the second edition of FEMA 154 (2002).

Table 3.10: Amplification of elastic demands for reductions of ductility due to vertical building irregularities

Ductility		**Amplification of elastic demand**	
Regular building	**Irregular building**	**Short periods**	**Long periods**
μ	**0.8 μ**	$R_{A_\mu}/R_{A_{0.8\mu}}$	$R_{V_\mu}/R_{V_{0.8\mu}}$
2	1.6	1.13	1.25
3	2.4	1.13	1.25
4	3.2	1.13	1.25
5	4.0	1.13	1.25
6	4.8	1.14	1.25
7	5.6	1.15	1.25
8	6.4	1.17	1.25
Average		**1.14**	**1.25**

Table 3.11: Amplification of elastic demands for reductions of ductility due to horizontal building irregularities

Ductility		**Amplification of elastic demand**	
Regular building	**Irregular building**	**Short periods**	**Long periods**
μ	**0.5 μ**	$R_{A_\mu}/R_{A_{0.8\mu}}$	$R_{V_\mu}/R_{V_{0.8\mu}}$
2	1.0	1.46	2
3	1.5	1.47	2
4	2.0	1.48	2
5	2.5	1.50	2
6	3.0	1.53	2
7	3.5	1.58	2
8	4.0	1.73	2
Average		**1.54**	**2**

The Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Structures (JBDPA 2001) prescribes a qualitative *irregularity* index q for first and second levels of screening, to modify the basic seismic index of existing reinforced concrete *buildings* due to plan eccentricities (horizontal *irregularities*) and variations in floor stiffness (vertical *irregularities*). *Irregularity* indexes are based on a grade index G_i, which reflects the severity of the different horizontal and vertical *irregularities*. Grade index G_i ranges from 0.8 for *buildings* with severe *irregularities* to 1 for regular *buildings*. As an alternative, G_i for certain *irregularities* is calculated as the inverse of amplification factors (F_e and F_s) prescribed in the Japanese *building* Standard Law (BCJ 1995). For *buildings* with eccentricities, a grade index G'_l is calculated as the inverse of eccentricity amplification factor F_e, while for *buildings* with stiffness or mass *irregularities*, grade index G'_n is calculated as the inverse of stiffness amplification factor F_s. Amplification factors F_e and F_s constitute a direct measure of the increase in a *building's* seismic demand due to horizontal and vertical *irregularities*. Maximum F_e and F_s values are 1.5 and 2, respectively.

Table 3.12 summarizes and compares the amplification of elastic seismic demand for different *seismic risk* screening methodologies. The predominant amplification factor for both vertical and horizontal *irregularities* is 2. The exceptions are the calculated factor of 1.25 for vertical *irregularities*, based on FEMA P-154 (third edition), and the factor of 1.5 for horizontal *irregularities* prescribed by the Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Structures and the second edition of FEMA 154 (2002). Based on this comparison, the maximum factor of 2 is selected to globally assess the seismic demand amplification of non-structural components in *buildings* with severe vertical *irregularities* or any horizontal *irregularity*. The calculated factor of 1.25, based on FEMA P-154, is selected for global scoring of non-structural components in buildings with moderate vertical *irregularities*.

Table 3.12: Amplification of seismic demands for different risk screening methodologies

Risk screening methodology	Vertical Irregularities		Horizontal Irregularities
	Severe	Moderate	
1993 NRC screening manual (structural index)	2/1.3[1]		1.5
The 1993 NRC screening manual (non-structural index)[2]	2/1.5		2/1.5
FEMA 154 (Second edition)	[3]		1.5[4]
Calculated based on FEMA P-154 (Third edition)[5]	1.25		2
Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Structures	2		1.5
Proposed methodology	2	1.25	2

[1] Factor of 2 for soft *storey* and 1.3 for other vertical *irregularities*.
[2] Same factor for vertical and horizontal *irregularities*. Factors of 2 and 1.5 correspond to editions prior to and after the NBC 1970, respectively.
[3] No amplification of demand or modification of capacity.
[4] Maximum amplification factor for a long period, calculated based on increase in seismic demand.
[5] Maximum amplification factors calculated for long periods based on rule of equal displacement and threshold for complete structural *damage*.

For a more detailed assessment, for each group of non-structural components, the factors in Table 3.13 are recommended. These factors are based on the maximum global factors for non-structural components controlled by short and long periods, corresponding to acceleration- and drift-sensitivity. Note that the critical components in Group 1, including Group 1a, are assumed to be drift-sensitive, while the components in Groups 2 and 3 are considered acceleration-sensitive.

Table 3.13: Amplification of elastic demands due to irregularities for non-structural component groups

Non-structural component group	Vertical		Horizontal
	Severe	Moderate	
All groups (global factors)	2	1.25	2
1) Architectural and structural components (categories 1 to 10)	2	1.25	2
1a) Architectural components forming part of a glazing system	2	1.25	2
2) Mechanical and electrical components (categories 11 to 17)	1.5	1.15	1.5
3) Other system components (categories 18 to 24)	1.5	1.15	1.5

Determination of NM_{VI} and NM_{HI}

Table 3.14 provides NM_{VI} and NM_{HI} values for global and group-specific scoring. These modifiers are determined with Eq. (3.13) as a function of the amplification of elastic demand factors provided in Table 3.13.

Table 3.14: NM_{VI} and NM_{HI} for non-structural component

Non-structural component group	NM_{VI}		NM_{HI}
	Severe	Moderate	
All groups (global seismic risk)	-8	-3	-8
1) Architectural and structural components (categories 1 to 10)	-8	-3	-8
1a) Architectural components forming part of a glazing system	-8	-3	-8
2) Mechanical and electrical components (categories 11 to 17)	-5	-2	-5
3) Other system components (categories 18 to 24)	-5	-2	-5

3.6.2.3 Pounding modifier

Pounding of adjacent structures or adjacent *building* parts and components due to insufficient separation can potentially increase seismic demand and *damage* of non-structural components, specifically at the zones of impact (Filiatrault et al. 1994). Previous studies and post-earthquake reconnaissance reports recognize *pounding* as a source of increases in floor accelerations and undesirable *damage* of non-structural components (Kasai et al. 1992, Anagnosotouplos 1996; and Dhakal et al. 2016). To prevent *damage*, non-structural components should resist the amplified seismic forces and accommodate additional displacements resulting from *pounding*. While higher seismic forces mostly affect acceleration-sensitive components, larger displacements are critical for drift-sensitive components.

The NBC 2015 and other seismic *building* codes, including ASCE/SEI 7-16, ASCE/SEI 41-16, EN 1998-1:2004 and NZS 4219:2009, prescribe minimum separations of adjacent structures as a function of displacements and *building* height. These requirements aim to prevent *pounding* of adjacent *buildings* but do not address probable impact of non-structural components with the structure or with other non-structural components. Similarly, the majority of the existing *seismic risk* methodologies consider the effect of *pounding* on *building* structures but not the effect on non-structural components.

The non-structural *pounding* modifier, NM_{PO}, is determined from the increase in the *building's* seismic demand due to *pounding* of adjacent *building* structures. NM_{PO} does not consider *pounding* of adjacent non-structural components for global *seismic risk*. This requires specific detailed screening for each component in the *building* following component-specific screening methodologies such as the CSA S832-14 standard.

<u>Pounding factor and amplification of seismic demand</u>

CSA S832-14 is the only *seismic risk* assessment methodology known by the authors that includes the effect of *pounding* in the *seismic risk* evaluation of non-structural components. This standard scores *pounding* based on two levels of separation adequacy between adjacent non-structural components and between non-structural components and the *building* structure. Rating scores of 1 and 10 are assigned to adequate and questionable (or inadequate) *building* separations, respectively. Although these rating scores are a qualitative measure of the increase in seismic vulnerability, they do not reflect the increase in seismic demand or the change in seismic response due to additional forces resulting from *pounding*. The two levels of separation adequacy correspond to low and severe vulnerability.

For *building* structures, the third edition of FEMA P-154 determines *pounding* modifiers for high and moderate severity *pounding*, based on vertical *irregularity* modifiers that do not apply to non-structural components. For high severity *pounding* in adjacent *buildings* with split levels and different heights, the *pounding* modifier corresponds to 100% of the average of the *severe vertical irregularity* modifiers for all FEMA *model building types*. For moderate severity *pounding* in *buildings* at the end of a row of three or more *buildings*, the *pounding* modifier corresponds to 50% of the average of the *severe vertical irregularity* modifiers for all FEMA *model building types*. For low severity *pounding*, the *pounding* modifier is zero.

The screening methodology in *Level 2 – SQST* considers four types of pounding and corresponding score modifiers for structural and non-structural component seismic scoring, based on the third edition of FEMA P-154. (Details of the criteria triggering *pounding* are provided in Section 2.5.8.) The types of pounding are as follows:

- Type 1: pounding that is not classified as Type 2, 3, or 4 (low severity pounding).
- Type 2: building at the end of the block (moderate severity pounding).
- Type 3: building with floors that do not align vertically within 0.6 m (high severity pounding).
- Type 4: building with height difference of two storeys or more (high severity pounding).

The criteria for moderate severity (Type 2) and high severity (Types 3 and 4) *pounding* are similar to those in the third edition of FEMA P-154 (2015). The score modifiers for moderate severity (Type 2) and high severity (Types 3 and 4) *pounding* correspond to 50% and 100% of the average of the *severe vertical irregularity* modifiers, respectively. In addition, the *Level 2 – SQST* considers low severity (Type 1) *pounding* for conditions other than those identified as moderate and high severity, and prescribes a score modifier corresponding to the 33% of the average of *severe vertical irregularity* modifiers for all *model building types*. Based on these criteria, *pounding* amplification factors F_{PO} applicable to non-structural components are determined as 2, 1.4, and 1.3 for high, moderate, and low severity, respectively, using the following equations:

$$F_{PO} = 1/10^{(NM_{VI}/26)} \quad \text{for high severity } \textit{pounding} \tag{3.15}$$

$$F_{PO} = 1/10^{(0.5NM_{VI}/26)} \quad \text{for moderate severity } \textit{pounding} \tag{3.16}$$

$$F_{PO} = 1/10^{(0.33NM_{VI}/26)} \quad \text{for low severity } \textit{pounding} \tag{3.17}$$

Table 3.15 provides the amplification factors due to *pounding* for the most critical *pounding* cases in the existing methodologies for *seismic risk* screening. It includes, CSA S832-14, FEMA P-154 (third edition), *Level 2 – SQST* for *building* structures, the Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Structures (*Building* Research Institute 2001), and the New Zealand Society for Earthquake Engineering Assessment and Improvement of the Structural Performance of *buildings* in Earthquakes methodologies (NZSEE 2013). Although the amplification factors capture the effect of *pounding* on the *seismic risk* of *building* structures and non-structural components, the differences in scoring approaches in the methodologies do not permit a direct comparison. North American methodologies, i.e. the 1993 NRC screening manual, FEMA P-154 (third edition), and CSA S832-14, prescribe factors greater than 1, which to some extent represent increases in seismic demands. CSA S832-14, however, follows a qualitative approach with a maximum factor of 10 that does not reflect the expected effect of *pounding* on seismic demand. The existing methodologies in Japan and New Zealand prescribe factors to reduce *buildings' seismic risk* index, reflecting the reduction in capacity to demand ratio, which is inversely proportional to the increase in demand. Given this approach, the amplification factors for the Japanese and New Zealand methodologies are determined as the inverse of the *pounding* factors. Disregarding the qualitative amplification factor of 10 in CSA S832-14, the most conservative amplification factor to increase seismic demand due to *pounding* corresponds to that based on *Level 2 – SQST* for *building* structures. Therefore, amplification factors of 2, 1.4 and 1.3, for severe, moderate, and low severity *pounding,* respectively, are used for global non-structural component scoring.

Table 3.15: Amplification factors due to pounding for different risk screening methodologies

Risk screening methodology	Amplification factor		
	Severe	Moderate	Low
CSA S832-14[1]	10	-	1
FEMA P-154/P-155[2]	2	1.5	1
Level 2 – SQST (*building* structures)[2]	2	1.4	1.3
1993 NRC screening manual (structural index)[3]	1.3	1.3	1.3
Japanese Standard for Seismic Evaluation of Existing Reinforced Concrete Structures[4]	1.1	1.05	1
New Zealand Assessment and Improvement of the Structural Performance of *Buildings* in Earthquakes[4]	1.25	1.1	1
Proposed methodology for global scoring	2	1.4	1.3

[1] Qualitative factors that do not reflect the increase in seismic demand.
[2] Amplification factors based on the *severe vertical irregularity* modifier.
[3] Amplification factors equal to *pounding* factors.
[4] Amplification factors calculated as the inverse of *pounding* factors.

Table 3.16 provides the global and group-specific amplification factors selected for the non-structural component risk assessment. The values of F_{PO} for different groups are calculated with Eqns. (3.15) to (3.17), based on the corresponding severe vertical modifiers in Table 3.14. For non-structural components in Groups 1 and 1a, F_{PO} values correspond to those for global non-structural component scoring of buildings. For non-structural component scoring for Groups 2 and 3, F_{PO} values are calculated 1.5, 1.25, and 1.15 for severe, moderate, and low *pounding*, respectively.

Table 3.16: Amplification factors due to pounding for non-structural component groups

Non-structural component group	Severe	Moderate	Low
All groups (global factors)	2	1.4	1.3
1) Architectural and structural components (categories 1 to 10)	2	1.4	1.3
1a) Architectural components forming part of a glazing system	2	1.4	1.3
2) Mechanical and electrical components (categories 11 to 17)	1.5	1.25	1.15
3) Other system components (categories 18 to 24)	1.5	1.25	1.15

Determination of NM_{PO}

Table 3.17 provides the global and group-specific NM_{PO}. Following the methodology for structural scoring in *Level 2 – SQST*, the *pounding* modifiers NM_{PO} for high, moderate, and low severity *pounding* are calculated using the *pounding* amplification factors in Table 3.16. The NM_{PO} values of -8, -5, and -3 for severe, moderate, and low *pounding*, respectively, are determined for global scoring. These NM_{PO} are the same as those for non-structural component Groups 1 and 1a. (Note that the value of NM_{PO} for moderate severity was slightly reduced to -5, from the calculated value of -4, to have a difference of at least 2 points with respect to NM_{PO} for low severity.) For the other non-structural components, Groups 2 and 3, NM_{PO} values are -5, -3, and -2, for severe, moderate, and low *pounding*, respectively.

Table 3.17: NM_{PO} for non-structural component

Non-structural component group	Severe	Moderate	Low
All groups (global seismic risk)	-8	-5	-3
1) Architectural and structural components (categories 1 to 10)	-8	-5	-3
1a) Architectural components forming part of a glazing system	-8	-5	-3
2) Mechanical and electrical components (categories 11 to 17)	-5	-3	-2
3) Other system components (categories 18 to 24)	-5	-3	-2

3.6.2.4 Building deterioration and age modifier

Deterioration and *ageing* due to lack of maintenance, such as spalling of masonry veneer and corrosion of mechanical connections, reduces non-structural components' capacity and seismic performance. *Deterioration* of individual non-structural components, however, is difficult to identify for the level of *seismic risk* screening in the *Level 2 – SQST*. Given this limitation, the *Level 2 – SQST* does not address the *deterioration* of non-structural components, which would require detailed evaluation of each component in the *building*. For non-structural components, the *Level 2 – SQST* provides a global score modifier based on *building deterioration* and *age*. This approach assumes that reductions in the structural capacity and energy dissipation due to material degradation amplify seismic demands, specifically on floors where the non-structural components are located. Furthermore, structural *damage* due to *deterioration* and *age* may induce *building irregularities* and associated increases in seismic demand. This effect has been recognized by the 1993 NRC screening manual, where *deterioration* is considered as a type of *building irregularity* for structural screening. Based on this approach, the increase in demand for non-structural components is determined as the inverse of the reduction in the *building* structure's capacity, established for the structural *deterioration* and *age* modifier. The criteria to trigger *deterioration* and *age* are discussed in Section 2.5.6. For significant *deterioration*, *Level 3 – Seismic Evaluation* of the non-structural components is required.

Building deterioration and age factor and amplification of seismic demand

Table 3.18 provides the scoring factors considering the effect of *building deterioration* and *age* on the *seismic risk* of *building* structures, in accordance with different codes and standards. The factors apply to the *seismic risk* screening of non-structural components, given the expected amplification in seismic demands resulting from *building deterioration*. Although not all existing codes and standards prescribe amplifications in seismic demands, equivalent amplification factors, based on the inverse of the reduction in strength and capacities, are calculated and presented in Table 3.18. The maximum amplification factor of 1.33, calculated as the inverse of the knowledge factor in ASCE/SEI 41-17, is selected to quantify the effect of *deterioration* and *age* on non-structural component response. This amplification factor is the same as the inverse of the capacity reduction factor k_d = 0.75, used to determine the structural *deterioration* and *age* modifier. The non-structural *deterioration* and *age* amplification factor is triggered by signs of *deterioration* and *building* age, in accordance with the criteria used to estimate structural *deterioration* and *age*, as provided in Section 2.5.6. To consider potential irregularity caused by *deterioration* and *age*, the amplification of seismic demands is multiplied by a factor of 1.25, which corresponds to the increase in demand for moderate vertical *irregularities* (Table 3.13). The effect of induced *irregularity* is not considered for *buildings* that are assessed with any type of *irregularity,* to avoid duplication of adverse effects. Therefore, global amplification parameters of 1.66 and 1.33 are considered for *deterioration* and *age* for irregular and regular *buildings*, respectively. These factors apply to the *seismic risk* screening of different non-structural component groups.

Table 3.18: Amplification factors due to deterioration and age

Code or Standard	Deterioration and/or age factor	Amplification factor
ASCE 41 (2013)	0.75[1]	1.33[2]
1993 NRC screening manual	1.3	1.3[3]
Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (JBDPA 2001)	1 (T < 20 years) 0.9 (T > 20 years) 0.8 (T > 30 years)	1[4] 1.11 1.25
Indian Guidelines for Seismic Evaluation and Strengthening of *Existing Buildings* (Rai 2005)	0.8[1]	1.25[2]

[1] Knowledge factor accounting for less confidence on structural capacity.

[2] Amplification factor based on the inverse of the product of reductions in strength and drift capacities.

[3] Amplification factor equal to *deterioration/age* factor.

[4] Amplification factors calculated as the inverse of the *deterioration/age* factors.

Determination of NM_{DA}

The global NM_{DA} values for regular and irregular *buildings* are determined as -6 and -3, respectively, based on the amplification factors of 1.66 and 1.33. Given that NM_{DA} is based on *deterioration* and *age* of *buildings*, the group-specific NM_{DA} values are the same as the global NM_{DA}.

3.6.3 Component response modifier

The component response modifier NM_{CR} is based on the formulation for S_p (Eq. (3.2)) prescribed by the NBC 2015. Three parameters that affect the response of the non-structural components are identified: flexibility (A_r), location in *building* elevation (A_x), and component response (R_p). NM_{CR}, however, only considers the effect of component response factor R_p. Assumed constant values of A_r (2.5 for critical flexible components) and A_x (2 for average elevation) are implicitly included in NM_{CR} to represent the global seismic risk of non-structural components in existing buildings. The average elevation of non-structural components is based on an assumed uniform distribution of non-structural components with potential risk to life safety. Since A_r and A_x are not considered in determining NM_{CR}, the minimum and maximum values of S_p (0.7 and 4, respectively) do not apply.

Component response factor

The NBC has evolved to improve the reliability and safety of non-structural components and their connections. As a result, the current formulation for the design seismic force in the NBC 2015 (Eq. (3.1)) explicitly prescribes more detailed seismic requirements for different categories of non-structural components (Assi and McClure 2015). Specifically, component response factor R_p has been introduced to determine the expected inelastic behaviour (ductility) of non-structural components and connections. The objective of the inelastic design is to improve seismic design and prevent brittle failure that may result in fall-out, overturning, or sliding of non-structural components. To reflect the effect of R_p on the seismic scoring of non-structural components, the component response modifier NM_{CR} is determined for two design periods: pre-2005 and post-2005. The pre-2005 period corresponds to the NBC design period where R_p was not prescribed for seismic design of non-structural components. The post-2005 period, on the other hand, corresponds to the NBC design period where the R_p factor is prescribed to determine non-structural components' seismic demand and acknowledge inelastic response.

Table 3.20 and Table 3.21 provide the R_p values for architectural and structural components, mechanical and electrical components, and other components, respectively. The R_p values are selected from the NBC 2005, which are, in general, the same values as those in the NBC 2010 and the NBC 2015. The exception is the R_P for elevators and escalators, pallet storage racks, as well as architectural components forming part of glazing systems (categories 22 to 24), which were introduced in the NBC 2015's non-structural component categorization. Between 2005 and 2015, non-structural categories 22 to 24 were designed in accordance with the NBC prescriptions for other rigid and flexible non-structural components corresponding to categories 18 to 21. Given this, categories 18 to 21 should be selected to select NM_{CR} for component-specific *seismic risk* screening between 2005 and 2010.

Table 3.19: Architectural and structural components

Category	**Component or equipment**[1]	$\boldsymbol{R_p}$
1	All exterior and interior walls except those in Categories 2 and 3	2.50
2	Cantilever parapet and other cantilever walls except retaining walls	2.50
3	Exterior and interior ornamentations and appendages	2.50
4	Floors and roofs acting as diaphragms	[2]
5	Towers, chimneys, smokestacks and *penthouses* when connected to or forming part of a *building*	2.50
6	Horizontally cantilevered floors, balconies, beams, etc.	2.50
7	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	2.50
8	Masonry veneer connections	1.50
9	Access floors	2.50
10	Masonry or concrete fences more than 1.8 m tall	2.50

[1] Categories based on the NBC 2005.

[2] The NBC 2015 does not prescribe a parameter for ductility of floors and roofs acting as diaphragms.

Table 3.20: Mechanical and electrical components

Category	Component or equipment[1]	R_p
11	Machinery, fixtures, equipment and tanks (including contents) that are rigid and rigidly connected that are flexible or flexibly connected	 1.25 2.50
12	Machinery, fixtures, equipment and tanks (including contents) containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids that are rigid and rigidly connected that are flexible or flexibly connected	 1.25 2.50
13	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a *building*	2.50
14	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a *building* containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids	2.50
15	Pipes, ducts (including contents)	3.00
16	Pipes, ducts (including contents) containing toxic or explosive materials, materials having a flash point below 38 °C or firefighting fluids	3.00
17	Electrical cable trays, bus ducts, conduits	5.00

Table 3.21: Other system components

Category	Component or equipment[1]	R_p
18	Rigid components with ductile materials and connections	2.50
19	Rigid components with non-ductile materials or connections	1.00
20	Flexible components with ductile materials and connections	2.50
21	Flexible components with non-ductile materials or connections	1.00
22	Elevators and escalators[2] Machinery and equipment that are rigid and rigidly connected Machinery and equipment that are flexible or flexibly connected Elevator rails	 1.25 2.50 2.50
23	Floor-mounted steel pallet storage racks[2]	2.50
24	Floor-mounted steel pallet storage racks on which toxics or explosive materials or materials having a flash point below 38 °C are stored[2]	2.50

[1] Categories based on the NBC 2005, except for categories 22, 23, and 24.

[2] Categories 22, 23, and 23 are based on the NBC 2015.

Determination of NM_{CR}

NM_{CR} is determined with Eq. (3.13) by defining F_i as the inverse of R_p. NM_{Rp} for both design periods, for global and group-specific scoring, are provided in Table 3.22. For both global and group-specific scoring, an R_p of 1 is used to determine NM_{CR} for the pre-2005 design period, while an R_p of 2.5 is considered for the post-2005 design period. These values are based on the characteristic R_p prescribed in the NBC for architectural non-structural components posing hazards to life safety. The corresponding NM_{CR} for the pre-2005 and post-2005 design periods are 0 and 10, respectively. The positive value of NM_{CR} for the post-2005 design period recognizes the expected improved response of non-structural components and connections designed for inelastic response with significant energy dissipation capacity.

Table 3.22: NM_{RP} for non-structural component

Non-structural component group	NM_{Rp}	
	Pre-2005	**Post-2005**
All groups (global seismic risk)	0	10
1) Architectural and structural components (categories 1 to 10)	0	10
1a) Architectural components forming part of a glazing system	0	10
2) Mechanical and electrical components (categories 11 to 17)	0	10
3) Other system components (categories 18 to 24)	0	10

In general, components that have an R_p of 2.5 represent a significant portion of non-structural components posing falling hazard to life safety. Furthermore, the intent of the *Level 2 – SQST* is to determine global score modifiers that reflect the improvement in post-2005 design, without the need to investigate the ductility of individual components. Thus, the NM_{Rp} values in Table 3.22 do not consider a few numbers of non-ductile, limited ductile, and highly ductile components. The R_p for architectural non-structural components in Group 1 and 1a is 2.5, with the exception of R_p of 1.5 for masonry veneer connections. A typical value of R_p for mechanical and electrical components is 2.5, which is reduced to 1.5 for rigid and rigidly connected components. Pipes and ducts, including contents, electrical cable trays, bus ducts, and conduits have R_p values greater than 2.5. Similarly, the characteristic R_p value for other system components is 2.5, with a few cases of R_p values of 1.0 and 1.25 for rigid components with non-ductile material or connections, as well as rigid machinery and equipment that are rigidly connected.

3.6.4 Design period modifier

The seismic provisions in the NBC have evolved to provide a more rational and accurate estimate of the design seismic forces for non-structural components. The design periods correspond to the NBC periods identified in Sections 3.3, i.e. 1) Pre-1953, 2) 1953-1975, 3) 1975-2005, and 4) post-2005. To account for the differences in design requirements, the design period modifier has been proposed for non-structural component scoring of buildings. Amplification of seismic demand for different design periods was previously considered by the 1993 NRC screening manual by prescribing non-structural indexes for pre-1970 and post-1970 *buildings*. The ratio of pre-1970 to post-1970 indices varies from 1 to 2, depending on the presence of *falling hazards* and the type of *building irregularities*.

Seismic demand modification factor

Given the qualitative *seismic risk* screening of non-structural components in the *Level 2 – SQST*, an approach based on the increase in seismic demand is followed to develop the non-structural design period modifier NM_{DP}, rather than the quantitative approach to determine the pre-code and post-benchmark modifiers for structural scoring. Note that in structural scoring, only three periods are identified, i.e., pre-code (pre-1953), pre-benchmark (between 1953 and 2005), and post-benchmark (2005 and later).

Table 3.23 provides seismic demand modification factors for different *building* design periods. The factors are based on the selection of a reference design period corresponding to the period from 1975-2005. Non-structural components designed before the reference period are penalized for potential poor seismic performance, while non-structural components designed after the reference period are credited for the significantly improved seismic design. The factor of 1, corresponding to the period from 1975-2005, recognizes the minimum non-structural design requirements based on horizontal force factor S_p. A maximum factor of 1.5 has been defined for the pre-1953 period to represent an increase of 50% in the reference S_p. The amplification factor of 1.25 for the 1953-1975 period is an intermediate value between those for the pre-1953 and 1975-2005 periods. The reduction factor of 0.7 for post-2005 non-structural components, corresponding to the inverse of the factor of 1.5 for the pre-1953 period, recognizes the improvements in seismic design in accordance with modern design standards.

Table 3.23: Modification factor due to building design year

Building design year	Modification factor
Pre-1953	1.5
1953-1975[1]	1.25
1975-2005[2]	1
Post-2005	0.7

[1] Excluding 1975
[2] Excluding 2005

Determination of NM_{DP}

Table 3.24 provides non-structural building design year modifier NM_{DP} for different design periods, calculated with Eq. (3.13). (Note that the calculated NM_{DP} for post-2005 was rounded up to 5.) The same magnitude with different signs for the NM_{DP} for pre-1953 and post-2005 represents two extreme ends of the seismic performance of non-structural components. The NM_{DP} values are applicable to both global and group-specific *seismic risk* assessments of non-structural components.

Table 3.24: Non-structural building design year modifier, NM_{DP}

Building design year	NM_{DP}
Pre-1953	-5
1953-1975[1]	-3
1975-2005[2]	0
Post-2005	5

[1] Excluding 1975
[2] Excluding 2005

3.6.5 Remaining occupancy time modifier

The non-structural *remaining occupancy time* modifier NM_{RO} recognizes the lower probability of non-structural components to experience the design earthquake event during the *building's* remaining *occupancy*. NM_{RO} grants a credit to non-structural components located in *buildings* with *remaining occupancy time* that is short enough to lower the design seismic spectra, providing a more accurate *seismic risk* screening of non-structural components located in *buildings* with shorter remaining *occupancy*.

Scaling factors

Remaining occupancy time is recognized by modifying the earthquake returning periods associated with different remaining *occupancy* years. Based on this approach, Fahti-Fazl et al. (2017) developed scaling *remaining occupancy time* factor κ to reduce seismic demand based on the return period for remaining *occupancy* of less than the original design life of 50 years. The κ factor is determined with Eq. (3.43).

Determination of NM_{RO}

Table 3.25 provides NM_{RO} for different remaining occupancies, based on the *remaining occupancy time* factor κ. NM_{RO} is rounded to 15 and 10 for 5 and 10 years of remaining *occupancy*, respectively. Note that NM_{RO} does not apply to non-structural components in *post-disaster buildings* with very high consequences of failure given that these non-structural components are essential to the provision of services in the event of an earthquake.

Table 3.25: Non-structural remaining occupancy modifier, NM_{RO}

Remaining occupancy	κ	NM_{RO}
5 years	0.28	15
10 years	0.44	10

3.7 Acceptable non-structural score threshold

3.7.1 Consequences of failure

The acceptable non-structural score threshold NS_{TH} accounts for life safety consequences of failure for non-structural components. Non-structural consequences of failure for global seismic risk screening are determined for different consequences of failure factors C_f, calculated as the product of the building importance category I_E, and the component factor of the most critical non-structural component C_P. While I_E considers the seriousness of the consequences of failure of the building structure supporting the non-structural components, C_p considers the risk associated with the failure of non-structural components. In the context of the *Level 2 – SQST* methodology, C_p focuses on *falling hazards* and hazardous materials that threaten the life of *building occupants* and passers-by. The C_p factor is applicable to non-structural components in buildings of low, normal, high, and *post-disaster* importance. Some non-essential and non-hazardous components may not be designed based on the *building's* I_E. In addition, some new non-structural components added to *existing buildings* may not be designed and installed in accordance with the *building's* original designated importance category. Given these reasons, and due to the conservative nature of the scoring procedure, the credit granted in the structural scoring system for I_E greater than 1.0 is not acknowledged in the non-structural scoring. However, acceptable thresholds are adjusted based on I_E to address the seriousness of consequences of failure corresponding to different importance categories.

Table 3.26 provides C_f for different building consequences of failure. For global seismic risk screening of existing buildings for their non-structural components, it is assumed that the non-structural consequences of failure are comparable to the structural consequences of failure and thus the consequence classification proposed by Fathi-Fazl and Lounis (2017) is applicable. Very Low, Low and Medium, High, and Very High consequences of failure levels are mapped to Low, Normal, High, and Post-Disaster importance categories, respectively. The values of C_f for different consequences of failure are determined for a C_p of 1.0 and 1.5 for non-hazardous and hazardous non-structural components, respectively. Determination of C_f disregards the minimum C_p of 0.7 in the NBC as it only applies to flat bottom tanks, including contents, attached directly to a floor at or below grade within the building; these are typically located in unoccupied areas and therefore pose minimum hazard to life safety.

Table 3.26: Consequence of failure factor, C_f

Consequences of failure	Component type	I_E	C_p	C_f
Very Low	Non-hazardous[1]	0.8	1.0	0.8
	Hazardous[2]	0.8	1.5	1.2
Low and Medium	Non-hazardous[1]	1.0	1.0	1.0
	Hazardous[2]	1.0	1.5	1.5
High	Non-hazardous[1]	1.3	1.0	1.3
	Hazardous[2]	1.3	1.5	2.0
Very High	Non-hazardous[1]	1.5	1.0	1.5
	Hazardous[2]	1.5	1.5	2.3

[1] Buildings containing no hazardous non-structural components.

[2] Buildings containing *hazardous* non-structural components (falling hazards or hazardous materials).

3.7.2 Basic non-structural score threshold

3.7.2.1 Threshold for non-structural components in seismic codes and design standards

In general, seismic codes and design standards recognize that non-structural components suffer negligible *damage* due to low-intensity earthquakes. The NBC 2005 and 2010 exempted the seismic analysis of structures and non-structural components located in sites design with a 5% damped spectral response acceleration at 0.2 second period, $S_a(0.2)$, that is less than or equal to 0.12 g. This exemption, however, was removed from the NBC 2015. The NBC 2015 provides a simplified design procedure for all structural and non-structural components in areas of low hazard with $I_E F_s S_a(0.2)$ less than 0.16 g and $I_E F_s S_a(2.0)$ less than 0.03 g. In addition, it prescribes simplified seismic design for six categories of architectural components including exterior and interior walls, cantilever parapets, other cantilever walls, exterior and interior ornamentations and appendages, towers, chimneys, smokestacks, and *penthouses*, with $I_E F_s S_a(0.2)$ less than 0.35 g. For *buildings* other than *post-disaster buildings*, seismically isolated *buildings*, and *buildings* with supplemental energy dissipation systems, where $I_E F_s S_a(0.2)$ is less than 0.35 g, the NBC 2015 exempts non-structural categories 6 through 22, consisting of mechanical, electrical, elevator equipment, pallet racks and other components from seismic design requirements. The $S_a(0.2)$ of 0.35 g corresponds to the upper bound of the Moderate *seismic zone*, which, in the *Level 2 – SQST,* is mapped to MMI VI+.

Similar damage thresholds are defined in other codes and standards. Based on the seismic requirements in the NBC 2010, the CSA S832 (2014) standard considers the *seismic risk* for non-structural components in *seismic zones* with $S_a(0.2)$ less than 0.12 as low priority. The CSA S832 (2014), determines a risk index based on the *seismicity* limit of $F_a S_a(0.2) = 0.35$ in the NBC 2010 to prescribe mitigation techniques for parapets, based on the height-to-thickness (h/d) ratios. ASCE 7-16 (2016) and FEMA E 74 (2012) prescribe exemptions for seismic design and *seismic risk* analysis of non-structural components in seismic design categories (SDC) A and B, which are comparable to MMI V and VI, with the exception of parapets supported by bearing walls or shear walls. Furthermore, the NIST standard (2011) states that all *buildings* (irrespective of their importance category) in SDC A, and all *buildings* with the exemption of essential *buildings* (equivalent to *post-disaster buildings*) in SDC B may not require screening and evaluation. Ground motions with MMI greater than VI are expected to cause hazardous *damage* to non-structural components. MMI of V and VI are lower than the MMI VI+ associated with $F_aS_a(0.2)$ = 0.35 g in the NBC 2015.

3.7.2.2 Determining the basic non-structural score threshold

The basic non-structural score threshold is determined by calculating the maximum non-structural *seismic risk* score, *NS*, for non-structural components with *Low-Medium consequences of failure,* corresponding to C_f = 1. The basic non-structural score threshold is therefore based on the following parameters:

- Low *seismic zone* where $F_aS_a(0.2)$ = 0.2 g (upper bound), NS_B = 64
- Site Class C with Fa = 1.0, NM_S = 0
- Critical non-structural component response (R_p = 1 for pre-2005 design)
- Maximum amplification of seismic demand in non-structural components due to structural condition and deficiencies (building type, component elevation, *building irregularities*, *pounding*, and *deterioration* and *age*), NM_{SR} = -20
- Critical non-structural component response (pre-2005 design), NM_{CR} = 0
- Pre-1953 design period, NM_D = -5
- Remaining occupancy of more than 10 years, NM_{RO} = 0

Table 3.27 summarizes the basic non-structural score threshold value calculations. The basic non-structural score threshold is 40.

Table 3.27: Calculation of basic non-structural score threshold

Description	**Score**
Basic non-structural score, NS_B	64
Site Class modifier, NM_{SC}	0
Structural response modifier, NM_{SR}	-20
Component response modifiers, NM_{CR}	0
Design period modifier, NM_{DP}	-5
Remaining occupancy time modifier, NM_{RO}	0
Non-structural score, *NS*	39 ≈ 40

3.7.3 Non-structural score thresholds for different consequences of failure

The non-structural score threshold NS_{TH} for different consequences of failure are determined by adding the threshold modifier $NS_{MTH} = 26\left[log_{10}\left(C_f/1.5\right)\right]$ to the basic non-structural threshold. (Note that the threshold modifier follows a logarithmic formulation similar to that used to determine the non-structural modifiers, except for the positive sign of the common logarithm.)

Table 3.28 provides NS_{TH} for the four consequence classes. NS_{TH} values have been adjusted to provide increasing thresholds in increments of 5. The basic NS_{TH} of 40 (rounded value for calculated NS_{TH} = 39) corresponds to C_f of 1.5 for hazardous components with Low-Medium consequences of failure. Lower and higher values of NS_{TH} are determined for other consequences of failure, based on the reference value of C_f = 1.5.

Table 3.28: Non-structural score thresholds for different consequence classes

Consequence class	**Component type**	C_f	NS_{TH}
Very Low	Non-hazardous[1]	0.8[1]	35
	Hazardous[2]	1.2[2]	40
Low-Medium	Non-hazardous[1]	1.0[1]	35
	Hazardous[2]	1.5[2]	40
High	Non-hazardous[1]	1.3[1]	40
	Hazardous[2]	2.0[2]	45
Very High	Non-hazardous[1]	1.5[1]	40
	Hazardous[2]	2.3[2]	45

[1] Buildings containing no hazardous non-structural components.

[2] Buildings containing *hazardous* non-structural components (falling hazards or hazardous materials).

3.7.4 Acceptance criteria

NS_{TH} conforms to the passing criteria in Table 3.29, based on *seismic zone*, *consequences of failure*, and *remaining occupancy time*. The passing criteria are based on the exemption criteria validated by Fathi-Fazl et al. (2018) in the Level 1 – PST. For Very Low and Low seismic zones specifically, Fathi-Fazl et al. (2018) compared the expected drift ratios and spectral accelerations for all building types with the corresponding HAZUS median drift ratio and peak floor acceleration thresholds. The median drift ratio proposed by HAZUS (FEMA 2012a) for slight damage to drift-sensitive non-structural components is 0.4%, while the median peak floor acceleration threshold for slight damage to acceleration-sensitive non-structural components is 0.2 g. These drift ratios are deemed to be appropriate for essential facilities, which are equivalent to buildings with Very High consequences of failure (FEMA 2012a).

Table 3.29: Exemption criteria of non-structural components based on seismicity, consequence class, and remaining occupancy

Seismic zone[1]		**Consequences of failure**			
		Very Low	**Low-Medium**	**High**	**Very High**
VL	Very Low $F_aS_a(0.2) \leq 0.10$	Pass			
L	Low $0.10 < F_aS_a(0.2) \leq 0.20$	Acceptable		Acceptable if: $n \leq 10$ years[2]	
M	Zone 3: Moderate $0.10 < F_aS_a(0.2) \leq 0.35$	Acceptable if: $n \leq 10$ years[2]		Acceptable if: $n \leq 5$ years[2]	
MH	Moderately High $0.10 < F_aS_a(0.2) \leq 0.75$	Acceptable if: $n \leq 5$ years[2]			
H	High $0.10 < F_aS_a(0.2) \leq 1.15$	Not acceptable Seismic risk screening or seismic evaluation of specific components is required			
VH	Very High $F_aS_a(0.2) > 1.15$				

[1] Seismic zones correspond to site seismic categories for *Site Class* C ($F_a = 1.0$).

[2] Remaining *occupancy* is considered when there exists a signed document from the owner or investor to confirm that the *building* is scheduled to be officially decommissioned or sold within a prescribed period of time.

3.7.5 Verification of passing criteria

Table 3.30 to Table 3.32 provide the *seismic risk* scoring for critical non-structural components for different *seismic zones* and *remaining occupancy times*. Critical non-structural components refer to those components that have maximum detrimental structural and non-structural effects (reflected via applicable non-structural *seismic risk* modifiers). In order to assess the passing criteria in the *Level 2 – SQST* in accordance with Very Low and Low seismic zones, modifiers corresponding to *Site Class C* are used. (The use of *Site Class* modifiers other than the one for *Site Class* C results in lower or higher non-structural scores, depending on the *building's Site Class*.) The remaining *occupancy* modifier does not apply to *buildings* with Very High consequence. The thresholds for all *consequences of failure* correspond to buildings containing hazardous non-structural components. In addition, the scores for non-structural components in *buildings* with Very High consequences of failure do not consider the positive remaining *occupancy* modifier. The scores in Table 3.30 to Table 3.32 indicate that acceptable non-structural component risk is in good agreement with the acceptance criteria in Table 3.29. (The "not acceptable" non-structural *seismic risk* scores are highlighted in grey.)

Table 3.30: Non-structural scores for critical pre-1953 non-structural components with remaining occupancy of more than 10 years

Seismic zone[1]	$S_a(0.2)$	Score	Threshold for consequences of failure[2]			
			Very Low $NS_{TH} = 40$	Low-Medium $NS_{TH} = 40$	High $NS_{TH} = 45$	Very High $NS_{TH} = 45$
Very Low	0.052	55	Acceptable	Acceptable	Acceptable	Acceptable
	0.07	51	Acceptable	Acceptable	Acceptable	Acceptable
	0.085	49	Acceptable	Acceptable	Acceptable	Acceptable
	0.1	47	Acceptable	Acceptable	Acceptable	Acceptable
Low	0.13	44	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.15	43	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.18	41	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.2	40	Acceptable	Acceptable	Not acceptable	Not acceptable
Moderate	0.24	37	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.28	36	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.32	34	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.35	33	Not acceptable	Not acceptable	Not acceptable	Not acceptable
Moderate High	0.45	30	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.55	28	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.65	26	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.75	25	Not acceptable	Not acceptable	Not acceptable	Not acceptable
High	0.85	23	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.95	22	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.05	21	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.15	20	Not acceptable	Not acceptable	Not acceptable	Not acceptable
Very High	1.3	18	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.45	17	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.6	16	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.73	15	Not acceptable	Not acceptable	Not acceptable	Not acceptable

[1] Seismic zone corresponding to *Site Class* C.
[2] NS_{TH} for hazardous non-structural components (falling hazard or hazardous materials).

Table 3.31: Non-structural scores for critical pre-1953 non-structural components with remaining occupancy of 10 years or less

Seismic zone[1]	$S_a(0.2)$	Score[2]	Score[3]	Threshold for consequences of failure[4]: Very Low NS_{TH} = 40	Low-Medium NS_{TH} = 40	High NS_{TH} = 45	Very High NS_{TH} = 45
Very Low	0.052	65	55	Acceptable	Acceptable	Acceptable	Acceptable
	0.07	61	51	Acceptable	Acceptable	Acceptable	Acceptable
	0.085	59	49	Acceptable	Acceptable	Acceptable	Acceptable
	0.1	57	47	Acceptable	Acceptable	Acceptable	Acceptable
Low	0.13	54	44	Acceptable	Acceptable	Acceptable	Not acceptable
	0.15	53	43	Acceptable	Acceptable	Acceptable	Not acceptable
	0.18	51	41	Acceptable	Acceptable	Acceptable	Not acceptable
	0.2	50	40	Acceptable	Acceptable	Acceptable	Not acceptable
Moderate	0.24	47	37	Acceptable	Acceptable	Acceptable	Not acceptable
	0.28	46	36	Acceptable	Acceptable	Acceptable	Not acceptable
	0.32	44	34	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.35	43	33	Acceptable	Acceptable	Not acceptable	Not acceptable
Moderate High	0.45	40	30	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.55	38	28	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.65	36	26	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.75	35	25	Not acceptable	Not acceptable	Not acceptable	Not acceptable
High	0.85	33	23	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.95	32	22	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.05	31	21	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.15	30	20	Not acceptable	Not acceptable	Not acceptable	Not acceptable
Very High	1.3	28	18	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.45	27	17	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.6	26	16	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.73	25	15	Not acceptable	Not acceptable	Not acceptable	Not acceptable

[1] Seismic zone corresponding to *Site Class* C.
[2] Score for Very Low, Low-Medium, and *High consequences of failure.*
[3] Score not considering remaining *occupancy* for *Very High consequences of failure.*
[4] NS_{TH} for hazardous non-structural components (falling hazard or hazardous materials).

Table 3.32: Non-structural scores for critical pre-1953 non-structural components with remaining occupancy of five years or less

Seismic zone[1]	$S_a(0.2)$	Score[2]	Score[3]	Threshold for consequences of failure[4]: Very Low NS_{TH} = 40	Low-Medium NS_{TH} = 40	High NS_{TH} = 45	Very High NS_{TH} = 45
Very Low	0.052	70	55	Acceptable	Acceptable	Acceptable	Acceptable
	0.07	66	51	Acceptable	Acceptable	Acceptable	Acceptable
	0.085	64	49	Acceptable	Acceptable	Acceptable	Acceptable
	0.1	62	47	Acceptable	Acceptable	Acceptable	Acceptable
Low	0.13	59	44	Acceptable	Acceptable	Acceptable	Not acceptable
	0.15	58	43	Acceptable	Acceptable	Acceptable	Not acceptable
	0.18	56	41	Acceptable	Acceptable	Acceptable	Not acceptable
	0.2	55	40	Acceptable	Acceptable	Acceptable	Not acceptable
Moderate	0.24	52	37	Acceptable	Acceptable	Acceptable	Not acceptable
	0.28	51	36	Acceptable	Acceptable	Acceptable	Not acceptable
	0.32	49	34	Acceptable	Acceptable	Acceptable	Not acceptable
	0.35	48	33	Acceptable	Acceptable	Acceptable	Not acceptable
Moderate High	0.45	45	30	Acceptable	Acceptable	Acceptable	Not acceptable
	0.55	43	28	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.65	41	26	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.75	40	25	Acceptable	Acceptable	Not acceptable	Not acceptable
High	0.85	38	23	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	0.95	37	22	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.05	36	21	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.15	35	20	Not acceptable	Not acceptable	Not acceptable	Not acceptable
Very High	1.3	33	18	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.45	32	17	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.6	31	16	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.73	30	15	Not acceptable	Not acceptable	Not acceptable	Not acceptable

[1] Seismic zone corresponding to *Site Class* C.
[2] Score for Very Low, Low-Medium, and *High* consequence classes.
[3] Score not considering remaining *occupancy* for *Very High* consequence class.
[4] NS_{TH} for hazardous non-structural components (falling hazard or hazardous materials).

Table 3.33 and Table 3.34 provide the worst- and best-case scenarios for *seismic risk* scoring for post-2005 non-structural components in different *seismic zones* and *remaining occupancy times* of more than 10 years. The worst-case scenario corresponds to the maximum detrimental effect of non-structural *seismic risk* modifiers for post-2005 design, while the best-case scenario corresponds to conforming non-structural components located in a *building* that is structurally sound. The results reflect the improvements in seismic performance of non-structural components designed in accordance with modern Canadian seismic provisions.

Table 3.33: Non-structural scores for critical non-structural components in post-2005 buildings with remaining occupancy of more than 10 years

Seismic zone[1]	$S_a(0.2)$	Score	Threshold for consequences of failure[2]			
			Very Low NS_{TH} = 40	Low-Medium NS_{TH} = 40	High NS_{TH} = 45	Very High NS_{TH} = 45
Very Low	0.052	75	Acceptable	Acceptable	Acceptable	Acceptable
	0.07	71	Acceptable	Acceptable	Acceptable	Acceptable
	0.085	69	Acceptable	Acceptable	Acceptable	Acceptable
	0.1	67	Acceptable	Acceptable	Acceptable	Acceptable
Low	0.13	64	Acceptable	Acceptable	Acceptable	Acceptable
	0.15	63	Acceptable	Acceptable	Acceptable	Acceptable
	0.18	61	Acceptable	Acceptable	Acceptable	Acceptable
	0.2	60	Acceptable	Acceptable	Acceptable	Acceptable
Moderate	0.24	57	Acceptable	Acceptable	Acceptable	Acceptable
	0.28	56	Acceptable	Acceptable	Acceptable	Acceptable
	0.32	54	Acceptable	Acceptable	Acceptable	Acceptable
	0.35	53	Acceptable	Acceptable	Acceptable	Acceptable
Moderate High	0.45	50	Acceptable	Acceptable	Acceptable	Acceptable
	0.55	48	Acceptable	Acceptable	Acceptable	Acceptable
	0.65	46	Acceptable	Acceptable	Acceptable	Acceptable
	0.75	45	Acceptable	Acceptable	Acceptable	Acceptable
High	0.85	43	Acceptable	Acceptable	Not acceptable	Not acceptable
	0.95	42	Acceptable	Acceptable	Not acceptable	Not acceptable
	1.05	41	Acceptable	Acceptable	Not acceptable	Not acceptable
	1.15	40	Acceptable	Acceptable	Not acceptable	Not acceptable
Very High	1.3	38	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.45	37	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.6	36	Not acceptable	Not acceptable	Not acceptable	Not acceptable
	1.73	35	Not acceptable	Not acceptable	Not acceptable	Not acceptable

[1] Seismic zone corresponding to *Site Class* C.
[2] NS_{TH} for hazardous non-structural components (falling hazard or hazardous materials).

Table 3.34: Non-structural scores for conforming non-structural components in post-2005 buildings with remaining occupancy of more than 10 years

Seismic zone[1]	$S_a(0.2)$	Score	Threshold for consequences of failure[2]			
			Very Low NS_{TH} = 40	Low-Medium NS_{TH} = 40	High NS_{TH} = 45	Very High NS_{TH} = 45
Very Low	0.052	85	Acceptable	Acceptable	Acceptable	Acceptable
	0.07	81	Acceptable	Acceptable	Acceptable	Acceptable
	0.085	79	Acceptable	Acceptable	Acceptable	Acceptable
	0.1	77	Acceptable	Acceptable	Acceptable	Acceptable
Low	0.13	74	Acceptable	Acceptable	Acceptable	Acceptable
	0.15	73	Acceptable	Acceptable	Acceptable	Acceptable
	0.18	71	Acceptable	Acceptable	Acceptable	Acceptable
	0.2	70	Acceptable	Acceptable	Acceptable	Acceptable
Moderate	0.24	67	Acceptable	Acceptable	Acceptable	Acceptable
	0.28	66	Acceptable	Acceptable	Acceptable	Acceptable
	0.32	64	Acceptable	Acceptable	Acceptable	Acceptable
	0.35	63	Acceptable	Acceptable	Acceptable	Acceptable
Moderate High	0.45	60	Acceptable	Acceptable	Acceptable	Acceptable
	0.55	58	Acceptable	Acceptable	Acceptable	Acceptable
	0.65	56	Acceptable	Acceptable	Acceptable	Acceptable
	0.75	55	Acceptable	Acceptable	Acceptable	Acceptable
High	0.85	53	Acceptable	Acceptable	Acceptable	Acceptable
	0.95	52	Acceptable	Acceptable	Acceptable	Acceptable
	1.05	51	Acceptable	Acceptable	Acceptable	Acceptable
	1.15	50	Acceptable	Acceptable	Acceptable	Acceptable
Very High	1.3	48	Acceptable	Acceptable	Acceptable	Acceptable
	1.45	47	Acceptable	Acceptable	Acceptable	Acceptable
	1.6	46	Acceptable	Acceptable	Acceptable	Acceptable
	1.73	44	Acceptable	Acceptable	Not acceptable	Not acceptable

[1] Seismic zone corresponding to *Site Class* C.
[2] NS_{TH} for hazardous non-structural components (falling hazard or hazardous materials).

The acceptable scores for critical non-structural components in pre-1953 buildings in Very Low and Low seismic zones (Table 3.30) demonstrate that the seismic risk scoring is in good agreement with the acceptance criteria in Table 3.29. The results in Table 3.30 confirm that the increase in seismicity results in lower non-structural component scores, and that the acceptance criteria are more stringent for High and Very High consequences of failure. The non-structural scores are not acceptable in the Moderate and higher seismic zones for all consequences of failure and in Low seismic zones for High and Very High consequences of failure. (Note that unacceptable scores are shaded in grey.) The results in Table 3.30, however, do not provide a quantitative measure of the non-structural components' seismic risk, specifically for individual non-structural components.

Therefore, detailed seismic risk screening or *Level 3 – Seismic Evaluation* of the specific non-structural components is recommended for unacceptable scores.

The acceptable scores for conforming non-structural components in post-2005 buildings in all seismic zones (Table 3.34) reflect the improvements in seismic performance of non-structural components designed in accordance with modern Canadian seismic provisions. The only exceptions to this trend were a couple of unacceptable scores for existing buildings with High and Very High consequences of failure in Very High seismic zones. These results are reasonable as there is still a low probability that very strong earthquakes would cause conforming non-structural components to fail, which could potentially affect the life safety of building occupants and passersby. The negative effect of non-structural failure could be aggravated in buildings with higher consequences of failure (i.e. High and Very High). The unacceptable scores do not necessarily mean that the non-structural components are deficient, but they signal the need for further investigation to evaluate the expected seismic performance and potential failure. This investigation is intended to be performed with a detailed seismic risk screening procedure or the *Level 3 – Seismic Evaluation Guidelines (SEG)*.

Seismic risk indexes, R, were calculated with the CSA S832-14 methodology to reflect the conditions of the cases. Maximum and minimum rating scores were used to determine building and non-structural component characteristics for pre-1953 critical and post-2005 conforming non-structural components, respectively. Values of $F_aS_a(0.2)$, based on reference Site Class C and seismic data in NBC 2015, were used to determine ground motion characteristics index. A threshold of $R = 16$ for negligible seismic risk level (mitigation not required) was used to define acceptable risk. This threshold is based on seismic design exemptions in NBC 2010 for buildings, other than post-disaster, located in areas of low seismicity. For both critical and conforming non-structural components, a consequence index of 10, corresponding to life safety, was assumed for all consequences of failure. [Note that the acceptance criteria proposed by the authors is consistent with the CSA S832-14 exemption criterion given that both methodologies aim to exempt non-structural components in low seismic zones.]

CSA S832-14 seismic risk indexes less than 16, corresponding to acceptable risk, were calculated for critical non-structural components in pre-1953 buildings with $F_aS_a(0.2)$ less than 0.18 g. For very low and low-medium consequences of failure, the proposed scoring system is in good agreement with the CSA S832-14 acceptable risk for $S_a(0.2)$ less than 0.18 g. For higher consequences of failure, the proposed scoring system is more stringent. A maximum CSA S832-14 seismic risk index of 10 was calculated for a maximum $F_aS_a(0.2)$ of 1.73 (upper bound of the very high seismic zone), based on minimum rating scores reflecting minimum vulnerability due to adequate non-structural and structural conditions. Given that the maximum seismic risk index was lower than the threshold seismic risk index of 16, the seismic risk for all seismic zones was deemed acceptable. This result confirms the acceptable risk determined with the proposed scoring system, specifically for very low and low-medium consequences of failure. For components in very high seismic zone with high and very high consequences of failure, the proposed scoring system estimated higher, not acceptable, risk.

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APPENDIX A STATE OF PRACTICE IN SEISMIC RISK SCREENING OF EXISTING BUILDINGS

A.1 Introduction

Building components can be divided into two categories: structural systems (i.e., gravity and lateral force-resisting systems) and non-structural components (e.g., architectural, mechanical, electrical, etc.). The latter can also be referred to as operational and functional components (OFCs) (CSA 2014). Structural systems are designed and constructed to carry and transfer all *building* loads (i.e., gravity and lateral) to the building's foundation without total or partial *building* failure or collapse, while non-structural components are not load bearing. A successful *seismic risk* screening of an existing building should primarily include the screening of both structural systems and non-structural components, given the potential risk to *life safety*.

This chapter presents a review of the current state-of-practice and current state-of-the-art on *seismic risk* screening of *existing buildings* around the world (i.e., Canada, U.S., and other countries). It includes two sections for *seismic risk* screening of: 1) structural systems, and 2) non-structural components.

A.2 Seismic risk screening of structural systems

A.2.1 State-of-practice

An overview of the available guidelines for *seismic risk* screening of *existing buildings* in North America (U.S. and Canada) and elsewhere is provided below.

A.2.1.1 North America

a) United States

- **FEMA 154**

In April 1988, the Applied Technology Council (ATC) developed the "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook", known as both the ATC-21 Report and FEMA 154. This Handbook presents a method whereby *buildings* can be rapidly identified, using a "*sidewalk survey,"* as seismically acceptable or potentially seismically hazardous, without performing any structural analysis calculations. The user would make decisions using a scoring system based on a seismic risk survey that would not take more than 30 minutes per *building*. FEMA 154 is the first of a two-volume publication containing a recommended methodology for rapid visual screening of *buildings* for potential seismic hazards.

The technical basis for the methodology in FEMA 154, including the scoring system and its development, is covered in the companion FEMA 155 Report entitled, "Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation".

The intent of FEMA 154 was to provide a standard rapid visual screening procedure to identify *buildings* that might pose a potentially serious risk of loss of life and injury, or of severe curtailment of community services, if a damaging earthquake were to occur. FEMA 154 would thus accomplish the objective of producing a document for local officials and practitioners outlining a cost-effective screening of *existing building* inventories in communities nationwide. The target audience for FEMA 154 was local *building* officials, professional engineers, registered architects, *building* owners, emergency managers, and interested citizens.

The first editions of both FEMA 154 and FEMA 155 were published in 1988. These documents were updated to their second and third editions in 2002 and 2015, respectively. In the following sections, a brief overview of all three editions is presented, including major updates in the second and third editions.

FEMA 154: First Edition – 1988

The first edition of FEMA 154 was published in 1988 and presented the rapid screening procedure (RSP), which consisted of inspecting a *building* from the exterior to quickly determine whether the *building* is likely adequate for potential earthquake forces, or whether there are reasonable doubts as to the *building*'s seismic performance. The purpose of the RSP is to determine whether the *building* should or should not be seismically evaluated in further detail to investigate its seismic adequacy.

The structural scoring system in FEMA 154 was based on the expected ground motion as defined in the National Earthquake Hazard Reduction Program (NEHRP) Map Areas, which were grouped into three categories according to *seismic* activity level, as follows:

- High (H): NEHRP map areas 5, 6, and 7
- Moderate (M): NEHRP map areas 3 and 4
- Low (L): NEHRP map areas 1 and 2

After identifying the *seismicity* region (i.e., H, M, or L) and selecting the appropriate data collection form (FEMA 1988), the Data Collection Form was then completed by gathering the following information (see example form in Figure A.1):

- **Basic information:** basic information about the *building* prior to the site visit (i.e., address, survey date, and identity of surveyor).
- **Photograph:** A general photo of the *building,* preferably showing two sides of the *building*.

- **Sketch:** A sketch of the *building* (i.e., plan and elevations), indicating dimensions, façade, structural materials, observed special features (such as cracks, seismic separation joint, roof tanks, cornices, etc.).

- **Building information:** Additional information about the *building*, including: number of *storeys*, *building age*, *occupancy* (e.g. residential, office, retail, wholesale/warehouse, light industrial, heavy industrial, public assembly such as auditoria or theatres, governmental), and an estimate of the number of persons in the *building* under normal *occupancy*. This information was considered of interest when determining mitigation priorities.

- **Non-structural falling hazard:** Identification of unbraced parapets, masonry cornices, chimneys, veneer, small cladding, overhangs especially on older structures.

- **Basic structural hazard (BSH) score:** Appropriate basic structural hazard (*BSH*) score according to primary structural material (i.e., wood, steel, concrete, precast, reinforced masonry or unreinforced masonry) and 12 predefined *building type* categories (Table A.1).

  The *BSH* was calculated as the negative of the logarithm (base 10) of the probability (P) of *damage* (D) exceeding 60% of the *building* value, given a ground motion represented by the NEHRP effective peak acceleration, as shown in Eq. (A.1):

  $$BSH = -log_{10}[P(D \geq 60\%)] \tag{A.1}$$

  FEMA 154 developed the BSH scores from earthquake *damage*-related information, using *damage* factors (*DF*) from ATC-13 (1985), wherein *damage* factor is defined as the ratio of dollar loss to replacement value. The *BSH* scores varied between 1 and 8.5, with a high value representing a seismically strong *building* and a low value corresponding to a potentially weak or hazardous *building*. The *BSH* score reflected the likelihood of the *building* sustaining major *damage*, given its seismic environment (e.g., BSH score of 2 meant that the probability of major *damage* is 1 in 100). Major *damage* referred to that requiring repair in excess of approximately 60% of the *building* value. FEMA 154 selected sixty percent *damage* as the generally accepted threshold of major *damage*, the point at about which many structures are demolished rather than repaired, and the approximate lower bound at which there begins to be a significant potential for the *building* collapse (significant life safety threat).

  The *BSH* scores were appropriate for "average" *buildings* designed and built in California, subjected to seismic loadings appropriate for NEHRP map area 7. In regions where *building* practices differed significantly from California *building* practices, the BSH needed to be modified. This required expert opinion to apply the ATC-13 information to regions outside California.

- **Modifiers:** Performance modification factors (PMFs), also called modifiers, capturing the deviations from the normal structural practice or conditions, or having to do with the effects of soil amplification on the expected ground motion.

  FEMA 154 identified a number of factors that can cause a deviation in a structure's seismic performance for an "average" or "normal" structure. The *PMFs* were assigned values, based on judgement, such that when added or subtracted from *BSH* scores, the resulting structural score (S) (Eq. (A.2)) would approximate the negative of the logarithm (base 10) of probability of major *damage* given the presence of that factor. These factors are determined based on visual inspection.

  $$S = BSH \pm PMF \quad \text{(A.2)}$$

  FEMA 154 considered the following PMFs that were applicable to the rapid visual screening methodology:

  - High-rise
  - Poor condition
  - Plan irregularities
  - Vertical irregularities
  - Soft storey
  - Pounding
  - Large heavy cladding
  - Short column
  - Torsion
  - Post benchmark year, and
  - Soil profile

  FEMA 154 defined *PMFs* based on the collective opinion of the project engineering panel and other engineers in the U.S. (FEMA 155 1988).

- **Cut-off score:** Minimum score representing acceptable *seismic risk.*

  FEMA 154 suggested a cut-off score, classifying a *building* either as safe or as requiring a more in-depth examination. In general, a *building* with a structural score less than a cut-off score of approximately 2 was considered not to meet modern seismic criteria and the *building* required detailed investigation, unless the community considered the cost-benefit aspects of lower seismic safety.

Table A.1: Different building types (from FEMA 154 1988)

W	Wood frame *buildings*
S1	Steel moment-resisting frame *buildings*
S2	Braced steel frame *buildings*
S3	Light metal *buildings*
S4	Steel frame *buildings* with cast-in-place concrete shear walls
C1	Concrete moment-resisting frame *buildings*
C2	Concrete shear wall *buildings*
C3/S5	Concrete or steel frame *buildings* with unreinforced masonry infill walls
PC1	Tilt-up *buildings*
PC2	Precast concrete frame *buildings*
RM	Reinforced masonry
URM	Unreinforced masonry

ATC-21/ (NEHRP Map Areas 5,6,7 High)

Rapid Visual Screening of Seismically Hazardous Buildings

Scale:

Address ______________________________

______________________ Zip__________

Other Identifiers______________________

No. Stories ____________ Year Built ________

Inspector _______________ Date __________

Total Floor Area (sq. ft)________________

Building Name_________________________

Use__________________________________

(Peel-off label)

INSTANT PHOTO

OCCUPANCY	
Residential	No. Persons
Commercial	0–10
Office	11–100
Industrial	100+
Pub. Assem.	
School	
Govt. Bldg.	
Emer. Serv.	
Historic Bldg.	

Non Structural Falling Hazard ☐

DATA CONFIDENCE

* = Estimated, Subjective, or Unreliable Data

DNK = Do Not Know

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	C1 (MRF)	C2 (SW)	C3/S5 (URM INF)	PC1 (TU)	PC2	RM	URM
Basic Score	**4.5**	**4.5**	**3.0**	**5.5**	**3.5**	**2.0**	**3.0**	**1.5**	**2.0**	**1.5**	**3.0**	**1.0**
High Rise	N/A	-2.0	-1.0	N/A	-1.0	-1.0	-1.0	-0.5	N/A	-0.5	-1.0	-0.5
Poor Condition	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Vert. Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-0.5	-0.5	-1.0	-1.0	-0.5	-0.5
Soft Story	-1.0	-2.5	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-2.0	-2.0	-1.0
Torsion	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Large Heavy Cladding	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
Short Columns	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
Post Benchmark Year	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 & 8 to 20 stories	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												

COMMENTS

Detailed Evaluation Required?

YES NO

Figure A.1: Sample Data Collection Form (from FEMA 154 1988)

FEMA 154: Second edition – 2002

After the first edition of FEMA 154 was published, FEMA developed a series of documents on the seismic evaluation and rehabilitation of *existing buildings*. Furthermore, a vast amount of information has been developed since 1998, including:

1. new knowledge about the performance of *buildings* during damaging earthquakes, including the 1989 Loma Prieta and 1994 Northridge earthquakes;
2. new knowledge about seismic hazards, including updated national seismic hazard maps published by the U.S. Geological Survey in 1996;
3. other new seismic evaluation and *damage* prediction tools, such as the "Handbook for the Seismic Evaluation of Buildings – a Prestandard" (ASCE 1998) and HAZUS (1999), which is FEMA's tool for estimating potential loss from natural disasters;
4. experience from the widespread use of the original FEMA 154 *Handbook* by federal, state and municipal agencies, and others.

The development of the new edition of FEMA 154 consisted of the following tasks:

1. obtaining users' feedback;
2. reviewing seismic performance of *buildings*, including a detailed review of the HAZUS fragility curves and an effort to correlate the relationship between results from the use of both the FEMA 154 rapid screening tool and the FEMA 178 – NEHRP Handbook for the Seismic Evaluation of Existing Buildings, for use in evaluating school buildings;
3. organizing users' workshops to learn first-hand problems and successes of organizations that use FEMA 154 on *buildings* within their jurisdictions;
4. updating both the FEMA 154 and FEMA 155 documents to create the second edition.

For the second edition of FEMA 154 Handbook, the scoring system was revised and the Handbook was shortened to focus on facilitating implementation. During the decade after publication of the first edition of the FEMA 154 Handbook, the rapid visual screening (RVS) procedure was used by both private and governmental agencies on more than 70,000 *buildings* nationwide. Through this widespread application, knowledge was gained about who the likely RVS procedure users would be and why they would use it, how easy the Handbook was to use, and how accurate the procedure's scoring system was. The revised RVS procedure retained the original procedure's framework and approach, yet incorporated a revised scoring system compatible with the ground motion criteria in FEMA 310 Handbook for the Seismic Evaluation of Buildings (FEMA 1998), and the *damage* estimation data provided in FEMA-funded HAZUS *damage* and loss estimation methodology (FEMA 1999). Both the *BSH* scores and score modifiers were updated using analytical calculations and HAZUS fragility curves for the *building type*s considered in the RVS methodology. Similar to the first edition of FEMA 154, the inspection, data collection, and decision-making process typically occurred at the *building* site, taking an average of 15 to 30 minutes per *building* or up to an hour when access to the interior of the *building* was permitted.

The major improvements in the second edition of FEMA 154 can be summarized as follows:

- Guidance on planning and managing RVS surveys, including training of screeners and acquisition of data from assessor files and other sources in order to obtain more reliable information on the building's age, structural system, and *occupancy*;
- More guidance on identifying the seismic force resisting system (SFRS) in the field;
- How to perform interior inspection or pre-survey reviews of *building* plans to identify/verify the *building*'s SFRS;
- Updated *building* structural hazard (*BSH*) scores and score modifiers, derived from updated HAZUS fragility curves;
- Use of new seismic hazard information, compatible with seismic hazard criteria in FEMA 310 and other related documents;
- Revised Data Collection Form, providing space for documenting Soil Type, additional options for documenting *falling hazards*, and expanded list of *occupancy* types.

The second edition of FEMA 154 provided a Data Collection Form (Figure A.2) for three *seismicity* regions (i.e., High, Medium, and Low), yet the boundaries of the three specified *seismicity* regions was modified, reflecting new knowledge about earthquake ground shaking and a change in the average return period from 475-year (corresponding to ground motions with 10% probability of exceedance in 50 years) to 2475-year (corresponding to ground motions with 2% probability of exceedance in 50 years). Design spectral accelerations at short period of 0.2 second and long period of 1 second were used to determine *seismicity* regions rather than effective peak accelerations. The design values, which were related to two-thirds (2/3) of the maximum considered earthquake (*MCE*), were obtained from the U.S. Geological Survey website. The use of design values helped to link the FEMA 154 results to the FEMA 310 (FEMA P-155 2002).

After identifying the *seismicity* region (i.e., H, M, or L) and selecting the appropriate Data Collection Form (FEMA 2002), the Data Collection Form was completed. The major changes in each section of the second edition of data completion form (compared to the first edition) are described below.

- **Building information:** This section was similar to that in the original Data Collection Form, where the specified nine general *occupancy* classes were the same classes used in the first edition of FEMA 154. The *occupancy* classes were retained for consistency, ease of identification from the street, and adequate representation of the broad spectrum of *building* uses in the U.S., matching the Uniform *Building* Code (UBC 1997). Furthermore, the *occupancy* load in the second edition of FEMA 154 was defined in ranges of 1 to 10, 11 to 100, 101 to 1000, and more than 1000, whereas in the first edition, it was defined in ranges of 1 to 10, 11 to 100, and more than 100.

- **Non-structural falling hazard:** Compared to the first edition of FEMA 154, this section was refined. It included some sub-categories of non-structural components (i.e., unreinforced chimneys, parapets, cladding, and other).

- **Basic structural hazard (BSH) score:** In order to determine the *building's BSH* score on the Data Collection Form, FEMA 154 required the *building*'s lateral load resisting system to be determined. While the first edition of FEMA 154 specified 12 types of *buildings* (Table A.1), the second edition identified 15 *building type*s. The increase in the number of *building type*s was mainly due to the fact that wood *buildings* (W) and reinforced masonry *buildings* (RM) were refined to W1 and W2, and RM1 and RM2, respectively, as follows:

  - Light wood-frame residential and commercial *buildings* ≤ 465 m2 (5 000 sf) (W1)
  - Light wood-frame *buildings* larger than 465 m2 (5 000 sf) (W2)
  - Reinforced masonry *buildings* with flexible floor and roof diaphragms (RM1)
  - Reinforced masonry *buildings* with rigid floor and roof diaphragms (RM2)

  The second edition of FEMA 154 implemented a more rational approach to determine the *BHS* score. Furthermore, the *BSH* score for each *building type* was defined as the negative of the logarithm (base 10) of the *building's* probability of collapse given a ground motion corresponding to the maximum considered earthquake (*MCE*), as shown in Eq. (A.3), whereas in the first edition, the *BSH* score was calculated based on the probability of *damage* exceeding 60% of the *building* value, given a ground motion represented by the NEHRP effective peak acceleration:

  $$BSH = -log_{10}[P(COL|MCE)] \tag{A.3}$$

  where *P(COL|MCE)* corresponded to the *building's* probability of collapse given the maximum considered earthquake (*MCE*). The *BSH* scores varied from 1.6 to 7.4, with a high value representing a seismically strong *building* and a low value corresponding to a potentially weak or hazardous *building*.

- **Modifiers:** Similar to the first edition, there were a number of major factors that could significantly impact structural performance during earthquakes. The score modifiers (formerly known as performance modification factors in the first edition) were added or subtracted from *BSH* scores; the resulting structural score *S* in Eq. (A.4) provided an indication of the probability of collapse given the presence of that factor. These factors were determined based on visual inspection.

$$S = BSH \pm Ms \quad (A.4)$$

In the second edition, FEMA 154 considered the following list of factors (with applicable score modifiers) in the rapid visual screening methodology:

- Mid-rise (4-7 storeys) and High-rise (more than 7 storeys)
- Plan irregularities
- Vertical irregularities
- Pre-code
- Post-benchmark
- Soil Type

The score modifiers for *building* height, design and construction year, plan *irregularity*, and Soil Type were calculated using a procedure that consists of:

1. calculating scores based on those characteristics using the procedure for calculating *BSH* scores, and

2. differentiating those scores from the "base case" to produce score modifiers (FEMA 155 2002).

Because it is not possible to quantify score modifier for vertical *irregularity*, largely due to the wide variety of configurations and conditions that could cause vertical *irregularity*, values for this score modifier were based on engineering judgement. In High and Moderate *seismicity* regions, the score modifier for vertical *irregularity* was assigned values such that, if it was the only modifier selected during the rapid visual screening process, the final structural score *S* would be less than the cut-off level of 2.0, thereby requiring detailed seismic evaluation of the *building*. FEMA 154 employed an increased seismic load as a proxy (for simplicity and practicality reasons) to approximate the effect of plan *irregularity* given the difficulties to calculate a score modifier that could capture plan *irregularities*. The plan *irregularity* score modifier was developed using the procedure for calculating the *BSH* score and spectral accelerations increased by 50%. The score modifiers for *building* height (Mid-rise and High-rise) and *post-benchmark buildings* were calculated using prescribed parameter values for these characteristics, which were found in the HAZUS (1999). The score modifier for Soil Type was calculated using the procedure for calculating the *BSH* score and modified spectral accelerations accounting for Soil Type effect.

The second edition of FEMA 154 (2002) did not consider the poor condition and *pounding* score modifiers that were previously considered in the first edition. An additional *pre-code* score modifier was included to address the worse seismic performance of *buildings* designed and constructed before seismic codes were initially adopted and enforced.

The second edition of FEMA 154 included a section in the scoring form to report non-structural *falling hazards*, specifically heavy cladding designed and installed before the jurisdiction adopted seismic anchorage requirements. FEMA 154 considered that improperly anchored components may result in non-structural components falling off the *building* during an earthquake, causing major changes to the *building* stiffness and setting up plan *irregularities* or torsion.

- **Cut-off score:** The second edition of FEMA 154 specified the same cut-off score of 2.0 specified in the first edition.

Rapid Visual Screening of Buildings for Potential Seismic Hazards

FEMA-154 Data Collection Form

HIGH Seismicity

Address: ______________________________

______________________ Zip ____________

Other Identifiers ______________________

No. Stories ______________ Year Built ________

Screener ______________ Date ____________

Total Floor Area (sq. ft.) ______________

Building Name ______________________

Use ______________________

Scale:

PHOTOGRAPH

OCCUPANCY			SOIL	TYPE						FALLING HAZARDS			
Assembly	Govt	Office	Number of Persons	A	B	C	D	E	F	☐	☐	☐	☐
Commercial	Historic	Residential	0 – 10 11 – 100	Hard Rock	Avg. Rock	Dense Soil	Stiff Soil	Soft Soil	Poor Soil	Unreinforced Chimneys	Parapets	Cladding	Other: ________
Emer. Services	Industrial	School	101-1000 1000+										

BASIC SCORE, MODIFIERS, AND FINAL SCORE, *S*

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	**4.4**	**3.8**	**2.8**	**3.0**	**3.2**	**2.8**	**2.0**	**2.5**	**2.8**	**1.6**	**2.6**	**2.4**	**2.8**	**2.8**	**1.8**
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8

FINAL SCORE, *S*

COMMENTS

Detailed Evaluation Required

YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know

BR = Braced frame
FD = Flexible diaphragm
LM = Light metal

MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm

SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure A.2: Sample Data Collection Form (FEMA 154 2002)

FEMA P-154: Third edition – 2015

In 2015, the Applied Technology Council (ATC) developed the third and current edition of FEMA P-154. The objective of the third edition remained the same as its predecessors: to identify, inventory, and screen *buildings* that were potentially hazardous in case of seismic activity. Although some sections of the text remained unchanged from the second edition, the third edition identified several areas of necessary enhancement, and considered the evolution of computer-aided tools to implement the procedure more efficiently. Major enhancements in the third edition are summarized below.

- The Level 1 screening is similar to the procedure used in the second edition, with the same objectives and the same general level of expertise required from the screeners. There are five Data Collection Forms for five *seismicity* regions: Low, Moderate, Moderately High, High, and Very High.
- The Data Collection Form (Level 1) was reorganized to enhance usability (see Figure A.5).
- A Level 2 optional screening form (Figure A.6) was introduced to obtain additional information and a more accurate assessment without a substantial increase in effort or time (still a rapid visual screening tool, yet relying on further information collected by an experienced engineer/architect).
- The Level 2 optional screening is more detailed than Level 1, and requires greater expertise to complete, yet is still rapid and visual (requiring some additional time, typically around 5-15 minutes per *building*, if occurring at the same time as the Level 1 screening). The Level 2 screening collects information about additional structural features affecting risk and provides refined score modifiers. Score modifiers were determined based on the relative severity of conditions and Level 1 score modifiers. There are five versions of the form, one for each of the five *seismicity* regions.
- The structural scoring procedure developed by the California Office of Statewide Health Planning and Development (OSHPD), which was based on HAZUS AEBM (2012a), was adopted and modified to calculate *BSH* scores and Level 1 score modifiers. Score modifiers were calculated by modifying the values of seismic demand, structural capacity and response parameters, seismic fragility parameters and collapse factor, which were used to calculate *BSH* scores.
- The number of *seismicity* regions was expanded from three to five (i.e., regions of Low, Moderate, Moderately High, High, and Very High *seismicity*) to increase screening accuracy in higher *seismicity* regions. The *seismicity* regions were determined based on risk-targeted maximum considered earthquake (MCE_R) ground motions, rather than the two-thirds of *MCE* ground motions used in the second edition.

- Vertical *irregularities* were separated into "Severe" and "Moderate" *irregularities* to reduce the penalty when only moderate *irregularities* exist.
- Reference guides were provided to assist with identifying vertical and plan *irregularities* and guide screeners in determining the existence of any *irregularity* and to reduce ambiguity and the need for judgement.
- Two new *building type*s were added to the Data Collection Form (Level 1): large multi-unit, multi-*storey* wood frame residential (W1A), and manufactured housing (MH) *building type*s (total of 17 *building type*s).
- The screening procedure for non-structural hazards was enhanced.
- *Occupancy* classes were updated.
- *Pounding* and adjacency were taken into consideration on the Level 1 Data Collection Form.
- Enhanced guidance for screening *buildings* with additions was provided.
- *Redundancy*, *pounding*, and comprehensive retrofits were taken into consideration in the Level 2 Optional Data Collection Form.
- A minimum score was defined on the Level 1 Data Collection Form to address negative scores.
- An optional electronic method was provided, with the use of the rapid observation of vulnerability and estimation of risk (ROVER) software.
- Information on how to run an effective screening program was added.

The major changes in each section of the third edition of Level 1 Data Collection Form are as follows:

- **Building information:** This section was reorganized, and some additional information was added, which is summarized as follows:
    - Geographical coordinates (i.e., longitude and latitude), as well as site specific ground motion values (i.e., S_{MS} and S_{M1}) were added.
    - The number of *storeys* in the *building* was divided into below and above grade.
    - A box was added to the "*year built*" section, to be checked if it is estimated.
    - The code year when the *building* was designed was added.
    - A new section was added, requiring the "*year built*" information when an addition has been added to the original *building*.

- The "*occupancy*" section was slightly changed. Two *occupancy* classes were added (utility and warehouse). Furthermore, the historical and government *occupancy* classes are now called additional designations. A new designation, called "shelter" was added.
- The "*falling hazards*" section in the second edition was revised to "Exterior *falling hazards.*" The "cladding" falling hazard was renamed to "*heavy cladding* or *heavy veneer.*" Also a new class of *falling hazard* called "appendages" was added.
- A new section called "Geological Hazards" was added, covering *liquefaction*, *landslide*, and surface fault rupture hazards.
- A new section, called "Adjacency" was added, covering the *pounding* and *falling hazards* from taller adjacent *buildings*.
- There is a new section, called "*Irregularities,*" in which the existence of any vertical and plan *irregularity* can be noted. The type and severity of the vertical *irregularity* (seven types including: sloping site [severe for W1, and moderate for all other *building types*], weak/soft *storey* [severe], out-of-plane setback [severe], in-plane setback [moderate], short column/pier [severe], and split levels [moderate]), as well as the type of plan *irregularity* (five different types including: torsion, non-parallel systems, re-entrant corners, large diaphragm openings, and beams not aligned with columns) can be specified.

- **Basic score:** In the third edition of FEMA P-154, the basic score for each *building type* was defined as the negative of the logarithm (base 10) of the *building's* probability of collapse given the risk-targeted maximum considered earthquake (MCE_R) ground motions, rather than two-thirds of *MCE* ground motions used in the second edition. The basic scores vary between 0.9 to 6.2, with a high value representing a seismically good *building* and a low value corresponding to a potentially weak or hazardous *building*.

- **Score modifiers:** Similar to the second edition, there are a number of major factors that can significantly impact structural performance during earthquakes. The score modifiers are added or subtracted from basic scores; the resulting structural score S approximates the negative of the logarithm (base 10) of probability of collapse given the presence of that factor. Similar to the first and second editions, until and unless a community considers the cost-benefit aspects of seismic safety for itself, if a *building*'s structural score S is less than about 2, the seismic performance of that *building* may not meet modern seismic criteria and the *building* should be investigated in detail. In the third edition of FEMA 154, the *irregularities* are limited to vertical and plan *irregularities*, and the *pounding* effect does not have any score modifier in the Level 1 Data Collection Form, unless the Level 2 Optional Data Collection Form is being completed. It should be noted that in the third edition, a credit is given to *buildings* with structural *redundancy* and *buildings* with previous comprehensive retrofit (i.e., the entire seismic force-resisting system has been strengthened). This is achieved through a Level 2 Optional Data Collection Form (in which more information is collected about additional structural features affecting risk and refined score modifiers are provided) by assigning positive score modifiers of 0.3 and 1.4 respectively, and consequently enhancing the *building's* overall structural score. However, Level 2 optional screening is required to be performed by a civil or structural engineering professional, architect, or graduate student with a background in *building* seismic evaluation or design. Vertical and plan *irregularities*, as well as *pounding* effect are covered in detail on the Level 2 Optional Data Collection Form, and refined score modifiers are specified. More detailed observation of non-structural *falling hazards* is also included on the Level 2 form.

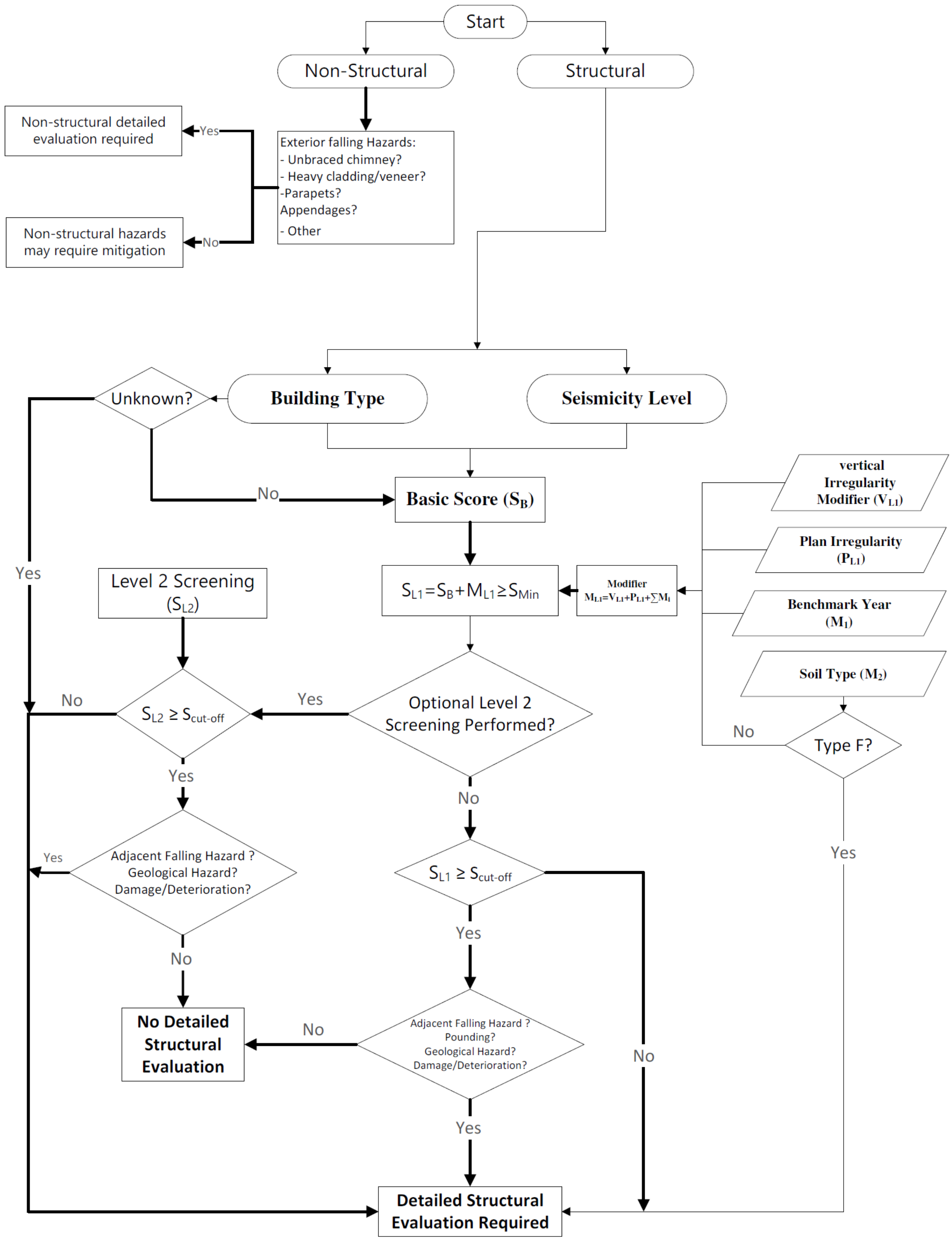

Figure A.3: Level 1 rapid visual screening procedure in FEMA P-154 (2015)

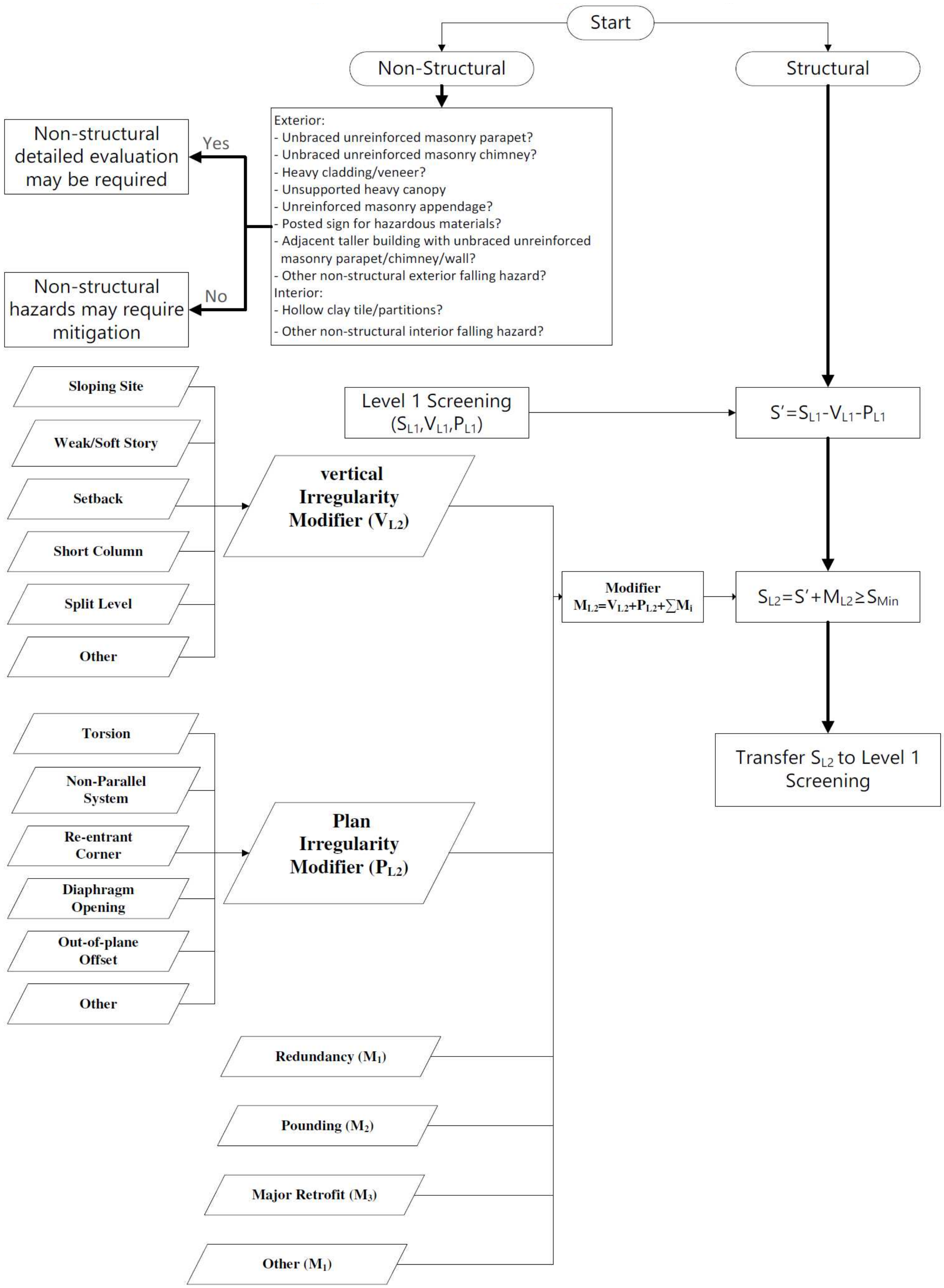

Figure A.4: Level 2 rapid visual screening procedure in FEMA P-154 (2015)

Rapid Visual Screening of Buildings for Potential Seismic Hazards **Level 1**
FEMA P-154 Data Collection Form **HIGH Seismicity**

PHOTOGRAPH

Address: ______________________
______________________ **Zip:** __________
Other Identifiers: ______________________
Building Name: ______________________
Use: ______________________
Latitude: __________ **Longitude:** __________
S_S: __________ S_1: __________
Screener(s): __________ **Date/Time:** __________

No. Stories: Above Grade: _____ Below Grade: _____ **Year Built:** _____ ☐ EST
Total Floor Area (sq. ft.): __________ **Code Year:** _____
Additions: ☐ None ☐ Yes, Year(s) Built: __________

Occupancy: Assembly | Commercial | Emer. Services | ☐ Historic | ☐ Shelter
Industrial | Office | School | ☐ Government
Utility | Warehouse | Residential, # Units: _____

Soil Type: ☐A Hard Rock | ☐B Avg Rock | ☐C Dense Soil | ☐D Stiff Soil | ☐E Soft Soil | ☐F Poor Soil | **DNK** *If DNK, assume Type D.*

Geologic Hazards: Liquefaction: Yes/No/DNK Landslide: Yes/No/DNK Surf. Rupt.: Yes/No/DNK
Adjacency: ☐ Pounding ☐ Falling Hazards from Taller Adjacent Building
Irregularities: ☐ Vertical (type/severity) __________
☐ Plan (type) __________
Exterior Falling Hazards: ☐ Unbraced Chimneys ☐ Heavy Cladding or Heavy Veneer
☐ Parapets ☐ Appendages
☐ Other: __________

COMMENTS:

SKETCH

☐ Additional sketches or comments on separate page

BASIC SCORE, MODIFIERS, AND FINAL LEVEL 1 SCORE, S_{L1}

FEMA BUILDING TYPE	Do Not Know	W1	W1A	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM	MH
Basic Score		**3.6**	**3.2**	**2.9**	**2.1**	**2.0**	**2.6**	**2.0**	**1.7**	**1.5**	**2.0**	**1.2**	**1.6**	**1.4**	**1.7**	**1.7**	**1.0**	**1.5**
Severe Vertical Irregularity, V_{L1}		-1.2	-1.2	-1.2	-1.0	-1.0	-1.1	-1.0	-0.8	-0.9	-1.0	-0.7	-1.0	-0.9	-0.9	-0.9	-0.7	NA
Moderate Vertical Irregularity, V_{L1}		-0.7	-0.7	-0.7	-0.6	-0.6	-0.7	-0.6	-0.5	-0.5	-0.6	-0.4	-0.6	-0.5	-0.5	-0.5	-0.4	NA
Plan Irregularity, P_{L1}		-1.1	-1.0	-1.0	-0.8	-0.7	-0.9	-0.7	-0.6	-0.6	-0.8	-0.5	-0.7	-0.6	-0.7	-0.7	-0.4	NA
Pre-Code		-1.1	-1.0	-0.9	-0.6	-0.6	-0.8	-0.6	-0.2	-0.4	-0.7	-0.1	-0.5	-0.3	-0.5	-0.5	0.0	-0.1
Post-Benchmark		1.6	1.9	2.2	1.4	1.4	1.1	1.9	NA	1.9	2.1	NA	2.0	2.4	2.1	2.1	NA	1.2
Soil Type A or B		0.1	0.3	0.5	0.4	0.6	0.1	0.6	0.5	0.4	0.5	0.3	0.6	0.4	0.5	0.5	0.3	0.3
Soil Type E (1-3 stories)		0.2	0.2	0.1	-0.2	-0.4	0.2	-0.1	-0.4	0.0	0.0	-0.2	-0.3	-0.1	-0.1	-0.1	-0.2	-0.4
Soil Type E (> 3 stories)		-0.3	-0.6	-0.9	-0.6	-0.6	NA	-0.6	-0.4	-0.5	-0.7	-0.3	NA	-0.4	-0.5	-0.6	-0.2	NA
Minimum Score, S_{MIN}		*1.1*	*0.9*	*0.7*	*0.5*	*0.5*	*0.6*	*0.5*	*0.5*	*0.3*	*0.3*	*0.3*	*0.2*	*0.2*	*0.3*	*0.3*	*0.2*	*1.0*

FINAL LEVEL 1 SCORE, $S_{L1} \geq S_{MIN}$:

EXTENT OF REVIEW

Exterior: ☐ Partial ☐ All Sides ☐ Aerial
Interior: ☐ None ☐ Visible ☐ Entered
Drawings Reviewed: ☐ Yes ☐ No
Soil Type Source: __________
Geologic Hazards Source: __________
Contact Person: __________

LEVEL 2 SCREENING PERFORMED?

☐ Yes, Final Level 2 Score, S_{L2} __________ ☐ No
Nonstructural hazards? ☐ Yes ☐ No

OTHER HAZARDS

Are There Hazards That Trigger A Detailed Structural Evaluation?
☐ Pounding potential (unless S_{L2} > cut-off, if known)
☐ Falling hazards from taller adjacent building
☐ Geologic hazards or Soil Type F
☐ Significant damage/deterioration to the structural system

ACTION REQUIRED

Detailed Structural Evaluation Required?
☐ Yes, unknown FEMA building type or other building
☐ Yes, score less than cut-off
☐ Yes, other hazards present
☐ No

Detailed Nonstructural Evaluation Recommended? *(check one)*
☐ Yes, nonstructural hazards identified that should be evaluated
☐ No, nonstructural hazards exist that may require mitigation, but a detailed evaluation is not necessary
☐ No, no nonstructural hazards identified ☐ DNK

Where information cannot be verified, screener shall note the following: EST = Estimated or unreliable data <u>OR</u> DNK = Do Not Know

Legend: MRF = Moment-resisting frame; BR = Braced frame; RC = Reinforced concrete; SW = Shear wall; URM INF = Unreinforced masonry infill; TU = Tilt up; MH = Manufactured Housing; LM = Light metal; FD = Flexible diaphragm; RD = Rigid diaphragm

Figure A.5: Sample Level 1 Data Collection Form (FEMA P-154 2015)

Optional Level 2 data collection to be performed by a civil or structural engineering professional, architect, or graduate student with background in seismic evaluation or design of buildings.

Bldg Name:	**Final Level 1 Score:**	*S_{L1} =*	*(do not consider S_{MIN})*
Screener:	**Level 1 Irregularity Modifiers:**	*Vertical Irregularity, V_{L1} =*	*Plan Irregularity, P_{L1} =*
Date/Time:	**ADJUSTED BASELINE SCORE:**	*S' = (S_{L1} – V_{L1} – P_{L1}) =*	

STRUCTURAL MODIFIERS TO ADD TO ADJUSTED BASELINE SCORE

Topic	**Statement** *(If statement is true, circle the "Yes" modifier; otherwise cross out the modifier.)*		**Yes**	**Subtotals**
Vertical Irregularity, V_{L2}	Sloping Site	W1 building: There is at least a full story grade change from one side of the building to the other.	-1.2	
		Non-W1 building: There is at least a full story grade change from one side of the building to the other.	-0.3	
	Weak and/or Soft Story *(circle one maximum)*	W1 building cripple wall: An unbraced cripple wall is visible in the crawl space.	-0.6	
		W1 house over garage: Underneath an occupied story, there is a garage opening without a steel moment frame, and there is less than 8' of wall on the same line (for multiple occupied floors above, use 16' of wall minimum).	-1.2	
		W1A building open front: There are openings at the ground story (such as for parking) over at least 50% of the length of the building.	-1.2	
		Non-W1 building: Length of lateral system at any story is less than 50% of that at story above or height of any story is more than 2.0 times the height of the story above.	-0.9	
		Non-W1 building: Length of lateral system at any story is between 50% and 75% of that at story above or height of any story is between 1.3 and 2.0 times the height of the story above.	-0.5	
	Setback	Vertical elements of the lateral system at an upper story are outboard of those at the story below causing the diaphragm to cantilever at the offset.	-1.0	
		Vertical elements of the lateral system at upper stories are inboard of those at lower stories.	-0.5	
		There is an in-plane offset of the lateral elements that is greater than the length of the elements.	-0.3	
	Short Column/ Pier	C1,C2,C3,PC1,PC2,RM1,RM2: At least 20% of columns (or piers) along a column line in the lateral system have height/depth ratios less than 50% of the nominal height/depth ratio at that level.	-0.5	
		C1,C2,C3,PC1,PC2,RM1,RM2: The column depth (or pier width) is less than one half of the depth of the spandrel, or there are infill walls or adjacent floors that shorten the column.	-0.5	
	Split Level	There is a split level at one of the floor levels or at the roof.	-0.5	
	Other Irregularity	There is another observable severe vertical irregularity that obviously affects the building's seismic performance.	-1.0	*V_{L2}* = ______
		There is another observable moderate vertical irregularity that may affect the building's seismic performance.	-0.5	*(Cap at -1.2)*
Plan Irregularity, P_{L2}	Torsional irregularity: Lateral system does not appear relatively well distributed in plan in either or both directions. *(Do not include the W1A open front irregularity listed above.)*		-0.7	
	Non-parallel system: There are one or more major vertical elements of the lateral system that are not orthogonal to each other.		-0.4	
	Reentrant corner: Both projections from an interior corner exceed 25% of the overall plan dimension in that direction.		-0.4	
	Diaphragm opening: There is an opening in the diaphragm with a width over 50% of the total diaphragm width at that level.		-0.2	
	C1, C2 building out-of-plane offset: The exterior beams do not align with the columns in plan.		-0.4	*P_{L2}* = ______
	Other irregularity: There is another observable plan irregularity that obviously affects the building's seismic performance.		-0.7	*(Cap at -1.1)*
Redundancy	The building has at least two bays of lateral elements on each side of the building in each direction.		+0.3	
Pounding	Building is separated from an adjacent structure by less than 1% of the height of the shorter of the building and adjacent structure and:	The floors do not align vertically within 2 feet. *(Cap total pounding modifiers at -1.2)*	-1.0	
		One building is 2 or more stories taller than the other.	-1.0	
		The building is at the end of the block.	-0.5	
S2 Building	"K" bracing geometry is visible.		-1.0	
C1 Building	Flat plate serves as the beam in the moment frame.		-0.4	
PC1/RM1 Bldg	There are roof-to-wall ties that are visible or known from drawings that do not rely on cross-grain bending. *(Do not combine with post-benchmark or retrofit modifier.)*		+0.3	
PC1/RM1 Bldg	The building has closely spaced, full height interior walls (rather than an interior space with few walls such as in a warehouse).		+0.3	
URM	Gable walls are present.		-0.4	
MH	There is a supplemental seismic bracing system provided between the carriage and the ground.		+1.2	
Retrofit	Comprehensive seismic retrofit is visible or known from drawings.		+1.4	*M* = ______
FINAL LEVEL 2 SCORE, S_{L2} = (S' + V_{L2} + P_{L2} + M) ≥ S_{MIN}:				*(Transfer to Level 1 form)*

There is observable damage or deterioration or another condition that negatively affects the building's seismic performance: ☐ Yes ☐ No
If yes, describe the condition in the comment box below and indicate on the Level 1 form that detailed evaluation is required independent of the building's score.

OBSERVABLE NONSTRUCTURAL HAZARDS

Location	**Statement** *(Check "Yes" or "No")*	**Yes**	**No**	**Comment**
Exterior	There is an unbraced unreinforced masonry parapet or unbraced unreinforced masonry chimney.			
	There is heavy cladding or heavy veneer.			
	There is a heavy canopy over exit doors or pedestrian walkways that appears inadequately supported.			
	There is an unreinforced masonry appendage over exit doors or pedestrian walkways.			
	There is a sign posted on the building that indicates hazardous materials are present.			
	There is a taller adjacent building with an unanchored URM wall or unbraced URM parapet or chimney.			
	Other observed exterior nonstructural falling hazard:			
Interior	There are hollow clay tile or brick partitions at any stair or exit corridor.			
	Other observed interior nonstructural falling hazard:			

Estimated Nonstructural Seismic Performance (*Check appropriate box and transfer to Level 1 form conclusions*)
- ☐ Potential nonstructural hazards with significant threat to occupant life safety → Detailed Nonstructural Evaluation recommended
- ☐ Nonstructural hazards identified with significant threat to occupant life safety → But no Detailed Nonstructural Evaluation required
- ☐ Low or no nonstructural hazard threat to occupant life safety → No Detailed Nonstructural Evaluation required

Comments:

Figure A.6: Sample Level 2 (Optional) Data Collection Form (FEMA P-154 2015)

b) Canada

- **1993 NRC screening manual**

In 1993, the National Research Council Canada (NRC) developed the Manual for Screening of Buildings for Seismic Investigation, hereinafter referred to as the 1993 NRC screening manual, for the purpose of rapid and inexpensive seismic screening to rank Canadian *buildings* in an inventory for further detailed seismic evaluation, which is compatible with Part 4 of the National Building Code of Canada (NBC) 1990. The 1993 NRC screening manual is meant to be the preliminary screening phase of a multi-phase procedure for identifying potentially hazardous *buildings* that might pose a risk of loss of life or injury, or severe curtailment of community services in the event of a damaging earthquake.

The NRC screening procedure is intended to prioritize potentially hazardous *buildings* for further detailed seismic evaluation. This prioritization is achieved through the computation of a scoring index, referred to as the Seismic Priority Index (*SPI*), reflecting parameters that can potentially affect a *building's*performance under a seismic event (i.e., *seismicity*, soil conditions, type of structure, structural *irregularities*, presence of non-structural hazards, *building age*, *building* importance, and *occupancy* characteristics). The parameters have been calibrated relative to the seismic requirements of the NBC 1990. This was based on Canadian seismic hazard values incorporated in the NBC 1985, as part of the third generation hazard maps developed by the Geological Survey of Canada (GSC), which remained in effect for both the NBC 1990 and 1995. In the NBC 1990, the effective *seismic zone* of the site of interest was calculated according to peak ground acceleration (PGA) and peak ground velocity (PGV) with a probability of exceedance of 10% in 50 years. The *SPI* is related to the degree to which the *building* is judged to deviate from NBC 1990's seismic requirements. Furthermore, the *benchmark NBC edition* for the screening is the NBC 1990. In general, higher *SPI* means greater priority for more in-depth seismic building investigation.

The NRC screening procedure was based on the first edition of FEMA 154. It incorporated interior *building* inspection and drawing review, which were not included in the first edition of FEMA 154. The NRC screening procedure also accounted for non-structural hazards, as well as *building* importance and *occupancy* characteristics. The NRC screening method requires no structural analysis calculations, and *building* inspection or drawing review takes an average of approximately one hour per *building*.

The 1993 NRC screening manual was meant to be qualitative in nature and implicitly based on seismic demand, rather than the probability of major *damage* in the first edition of FEMA 154 or probability of collapse in the second and third editions of FEMA 154. The purpose of the 1993 NRC screening procedure was to prioritize *buildings* in moderate and high *seismicity* regions and to rank them for detailed seismic evaluation. The *building's seismic risk* was represented by a seismic priority index (*SPI*), which was calculated by incorporating several parameters including *seismicity* (*A*), soil conditions (*B*), type of structure (*C*), structural *irregularities* (*D*), *building* importance and *occupancy* characteristics (*E)* and presence of non-structural hazards (*F*). These parameters are based on the differences in the requirements between the edition of the NBC used in the original design of the *building*, and the NBC 1990. Information such as the *year built* and applicable NBC edition are two key parameters for determining a *building's seismic risk*. Therefore, this information, as it relates to the *existing building's* design and construction practices, is tied directly to the individual scores for the above mentioned parameters. The *SPI* scoring system consists of two main components:

1) Structural index (*SI*), which is associated with possible risk to the *building* structure, and is calculated as:

$$SI = A \cdot B \cdot C \cdot D \cdot E \tag{A.5}$$

2) Non-structural index (*NSI*), which is associated with the risk of non-structural *building* components, and is calculated as:

$$NSI = B \cdot E \cdot F \tag{A.6}$$

The *SPI* is equal to the sum of the *SI* and *NSI*.

$$SPI = SI + NSI \tag{A.7}$$

The 1993 NRC screening manual suggests that *buildings* with: (i) *SPI* score that is less than 10 – Low priority; (ii) *SPI* between 10 and 20 – Medium priority; and (iii) *SPI* greather than 20 – High priority. A high index suggests that the *building* should be subjected to additional study by a professional engineer experienced in seismic design, while a low index indicates that the *building* is probably adequate. *Buildings* with *SPI* scores > 30 are considered potentially hazardous.

- **British Columbia, Canada**

In 2004, the Ministry of Education of British Columbia was involved in a program to evaluate the seismic vulnerability of 850 public schools located in high risk zones in the province of British Columbia. The evaluation was conducted by special-purpose rapid seismic screening software UBC-21 developed for this purpose. The *building* evaluation conducted by this tool incorporated five parameters that influence the seismic performance of structures. These parameters were: *seismic zone*, structural lateral resisting system, capacity, year of construction, and structural *irregularities*.

Based on these results, the BC Ministry of Education announced a $1.5 billion seismic mitigation program in 2004 with the goal of retrofitting all seismically vulnerable low-rise school *buildings*. Given the size of the project, the Ministry engaged the Association of Professional Engineers and Geoscientists of BC (APEGBC), with the collaboration of the Department of Civil Engineering of the University of British Columbia (UBC), to assist with implementing the seismic upgrade program for schools in BC. In 2006, due to the urgent need to start the retrofitting program, the interim Bridging Guidelines for the Performance-Based Seismic Retrofit of BC were created to provide APEGBC members with rational guidelines for seismic assessment and retrofit of BC schools. The interim Bridging Guidelines were replaced by the Seismic Retrofit Guidelines (1st edition). Both guidelines apply the same performance methodology. The 2nd edition of the Seismic Retrofit Guidelines was released in November 2013. The guidelines include enhanced information on *seismicity*, common types of school construction, and prioritizing vulnerable structural components. The seismic resistance criteria for specific construction can be easily determined by practitioners using the web-based tools provided in the guidelines. The 3rd edition (current edition) of the Seismic Retrofit Guidelines was released in July 2018.

A.2.1.2 Other countries

Seismicity, construction materials used, soil, *building type*s, as well as codes and standards differ from country to country. Thus, it is not a good practice to apply seismic screening guidelines that were developed for a specific country or region in another country with completely different seismic characteristics or *building* codes. Accordingly, several countries established their own guidelines for seismic screening and evaluation of *existing buildings* that can predict accurately *buildings* at risk. This sub-section provides a brief overview of RVS methods available in countries other than the U.S. and Canada.

a) New Zealand

The New Zealand Society for Earthquake Engineering (NZSEE) developed “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes” guidelines in 2006 (NZSEE, 2006) for structural assessment of *buildings* subjected to earthquakes in New Zealand. The basic aim of these guidelines was to evaluate the earthquake performance of *existing buildings* and to specify whether or not they are earthquake-prone.

The NZSEE guidelines include assessment procedures for *buildings* of various configurations and material types (e.g. reinforced concrete, steel, timber and unreinforced masonry) and methods for improving the performance of *buildings*. The first stage is the Initial Evaluation Procedure (IEP), in which a coarse screening is performed with limited resources to identify earthquake-prone *buildings*. The effects of any aspects of the *building* and/or its parts that might reduce *building* performance are qualitatively assessed in the IEP. *Buildings* that need detailed assessment of their earthquake performance are identified and *building* risk scores are obtained at the end of this stage. Only New Zealand Chartered Professional Engineers with a strong background in designing structures for seismic action should carry out the IEP. The second stage is a detailed assessment of earthquake performance and it is required whenever the initial evaluation indicates that the *building* is potentially hazardous. The detailed assessment gives a more specific and convincing evaluation based on which the final decision can be made; nevertheless, more detailed *building* characteristics and nonlinear static analyses to predict the structure response during seismic action are required.

In 2017, NZSEE developed "The Seismic Assessment of Existing Buildings" guidelines, which supersedes the 2006 NZSEE guidelines. The 2017 NZSEE guidelines consist of a qualitative Initial Seismic Assessment (ISA) method and a more extensive and quantitative Detailed Seismic Assessment (DSA) method. The Initial Seismic Assessment (ISA) method is intended to be the first step in the overall seismic assessment process to develop a holistic understanding of likely building seismic performance. An earthquake rating %*NBS* (percentage achieved compared with requirements for a new building) is used to indicate the earthquake performance of an *existing building*. A *building* with a %*NBS* of less than 34 is identified as earthquake-prone. A %*NBS* of 34 or greater means that the *building* is outside the requirements of the earthquake-prone provisions. A %*NBS* of 67 or greater indicates that the *building* should not be a significant earthquake risk. The Detailed Seismic Assessment (DSA) method aims to provide a more reliable and consistent outcome in terms of %*NBS* by quantifying the strength and deformation capacities of the various structural elements and by checking the *building* structural integrity against the loads/deformations that would be used for the design of a similar *building* on the same site. The DSA can be performed with different levels of detail depending on the assessment scope and objectives. A *building* is identified as an earthquake risk *building* if its %*NBS* is less than 67.

b) Japan

The evaluation of the seismic performance of existing low- and mid-rise reinforced concrete *buildings* was carried out in Japan since 1975 using the Japanese Seismic Index Method (Calvie et al. 2006 and Tischer 2012). In this procedure, the seismic performance index (I_s) is computed for each *storey* in each main direction to predict the *building's* seismic performance. The seismic performance index is determined as follows:

$$I_s = E_o.S_D.T \tag{A.8}$$

where,

E_o is the basic structural performance sub-index,

S_D is the structural design of the *building* sub-index, and

T is the time-dependent *deterioration* of the *building* sub-index.

The basic structural performance sub-index E_o is related to the ultimate strength index and ductility index with the failure mode, total number of *storeys* and position of *storey* being considered. The structural design of the *building* sub-index S_D is a scale for the influence of *irregularity*, stiffness and/or mass concentration of the *building*. The time-dependent *deterioration* of the *building* T is based on the data collected through a field inspection. The seismic performance index I_s, once found, is compared with the seismic judgment level I_{so} to specify the *building* seismic vulnerability level. The structure is considered as safe when $I_s > I_{so}$, while retrofitting or demolishing is required when $I_s < I_{so}$. Finally, when I_s is slightly smaller than I_{so}, a more detailed assessment is required using the next screening level or nonlinear dynamic analysis (Calvie et al. 2006). The seismic judgment level I_{so} can be obtained as follows:

$$I_{so} = E_s.Z.G.U \tag{A.9}$$

where,

E_s: is originally taken as 0.6 for the second and third level screening and 0.8 for the first;

Z : is a zone index used to modify the intensity of the ground motion at the site of the *building;*

G: is the ground index used to account for ground-*building* interaction, amplification in the top layer of ground or topographical effects;

U: is a usage index that may be seen as an importance factor concerned with the *building's* function.

According to Nanda and Majhi (2014), the seismic evaluation using the Japanese procedure is based on very few parameters and lacks clarity regarding *building* ranking based on a scoring system.

c) India

In 2002, the Structural Engineering Research Center (SERC) in Chennai, India, prepared a guideline document for the assessment of strength and performance/safety of *existing buildings* for both masonry and multi-*storey* reinforced concrete structures (Nanda and Majhi 2014). The guidelines also include recommendations on retrofitting schemes for *building* safety against earthquakes. According to this document, *buildings* are classified into five types, including: unreinforced masonry load-bearing wall *buildings*, reinforced masonry load-bearing wall *buildings* with stiff diaphragms, reinforced concrete moment frames, reinforced concrete frames with infill masonry shear walls, and reinforced concrete shear wall *buildings*. For each type, an *n*-factor ranging from 1 to 5 that represents the *building's* ductility and energy dispassion capacity is assigned. In order to suit Indian conditions, a modified FEMA 154 (2002) procedure is used in the first level of this procedure (rapid evaluation). Based on the *seismic zone*, age of the *building*, number of *storeys*, eccentricity and types of soil and foundations, a structural score S is determined. If this score is > 1 then a structural analysis is required (2^{nd} level). This level is very much similar to the FEMA 310 Tier 1 procedure.

d) Italy

The Italian Vulnerability Index Method is a scoring methodology in which the assessment is based on visual observations conducted through a field survey to identify the *building's* primary structural system and significant seismic deficiencies. The detailed description of the *building's* seismic deficiencies provided through this method may be used to estimate the *damage* for equivalent *building* models. In the first assessment level in this method, a vulnerability index for the *building* is assigned based on the type of resisting system and organization, resisting system quality, location and soil condition, diaphragms, plan and vertical configurations, connectivity between elements, low-ductility structural members, non-structural components and preservation state. The occurrence of an earthquake and associated *building damage* state is also assessed with a *damage* index. Finally, for different earthquakes, the vulnerability function is determined from the relationship between the vulnerability and *damage* indices. For a certain *building type*, the vulnerability function is affected by the quality of construction materials, construction practices, and code compliance.

e) Balkan region

The technical descriptions of the seismic mitigation measures for structures in the Balkan region are combined in a manual with a discussion on policy issues associated with the seismic mitigation progress. The post-earthquake and pre-earthquake assessment program is also discussed in the manual. This document is part of a bigger project for *building* construction under seismic conditions in the Balkan Region, carried out with the participation of the Governments of Bulgaria, Greece, Hungary, Rumania, Turkey and Yugoslavia, along with the United Nations Industrial Development Organization (UNDP/UNIDO). From a quality point of view, *buildings* are classified as good, acceptable or unclear according to this manual. A strength index R is obtained from the shear force capacity and required shear capacity. For simplicity, the axial load-moment interaction diagrams are used to determine the strength index R. The calculated strength index R is then compared to three pre-defined values of R. Based on the outcomes of this comparison, it is decided whether to strengthen the structure. Knowing the R value and estimated structural layout quality, the *building* is classified into five categories. The type of strengthening is then selected when the deformability and ductility requirements are checked. The remaining life of the evaluated structure can also be predicted using this method.

f) Greece

In 2002, the Earthquake Planning and Protection Organization (OASP) developed the Greek RVS OASP-0 method. This method is based on the first edition of FEMA 154. *Building* lateral load-resisting systems and structural materials are identified through visual screening. *Buildings* are then classified as one of the 18 predefined structural types and initial structural hazard scores are determined. Final scores are obtained by applying score modifiers to the basic structural hazard score. Score modifier values depend on *seismic zone*, weak *storey*, short columns and regular arrangement of masonry walls. A detailed investigation should be performed for *buildings* with final score ≤ 2. To accurately identify potentially hazardous *buildings*, the OASP developed the OASP-R method, which is based on the second edition of FEMA 154.

g) Switzerland

In the Swiss Society of Engineers and Architects (SIA) 2018 standard (SIA 2004), three stages are applied to evaluate the *seismic risk* of *buildings* investigated. The *building* plan is used and visual inspection is carried out in the first stage to roughly assess the structure's *seismic risk*. A detailed evaluation is applied for selected *buildings* in the second stage. Strengthening measures are described for a limited number of vulnerable *buildings* in the third stage (Achs and Adam 2012).

h) European Union

The European Committee for Standardization (CEN, French: Comité Européen de Normalisation) approved the European code procedure in 1995 as a prospective standard for provincial applications. The aim of the European code procedure is: to evaluate the seismic performance of existing structures, to describe the corrective measures required, and to set criteria for designing the strengthening process. In the evaluation processes, the seismic resistance of *damage*d or un*damage*d *buildings* is verified for both seismic and non-seismic actions. A computational verification is also conducted at the component level, in which all cross-sections are verified. The post-yield deformations should always be higher than the corresponding demand values, and the predicted *damage* level for structural and non-structural components must be kept within acceptable limits (Nanda and Majhi 2014).

i) Turkey

In Turkey, several preliminary assessment procedures have been proposed by different researchers. Hassan and Sozen proposed a priority index method in 1997, where the *building's* priority index is used to define its level of vulnerability. The priority index, which is a function of indices for walls and columns, is calibrated using *damage* data. The wall index and column index are determined by dividing the area of walls and area of columns by the total *floor area*, respectively. In 2004, Yakut proposed a capacity index method that takes into account the orientation, size and material properties of the lateral load-resisting system, quality of workmanship and materials, architectural features as well as plan *irregularities*. The capacity index classifies *buildings* at either low or high risk. Ozdemir et al. (2005) proposed the seismic safety screening method (SSSM), which was adapted from the Japanese seismic index method. In this method, the *building* is judged as safe if the seismic index value is greater than the demand index value, otherwise further vulnerability assessment is needed. Nonlinear static analyses of 12 *buildings* in Istanbul, Turkey, were used to calibrate this method for Turkish *buildings* (Ozdemir et al. 2005 and Calvie et al. 2006). A simple screening method was proposed by Sucuoglu et al. in 2007 to evaluate three- to six-*storey* substandard concrete *buildings*. Based on the number of *storeys* and the *seismic zone,* a basic score is assigned and the *building* is classified in one of the four predefined *damage* grads. The existence of soft *storeys*, apparent quality and heavy overhangs are considered to predict the vulnerability coefficient. This procedure was calibrated with field data collected from 454 *buildings* after the 1999 Duzce Earthquake (Perrone *et al.* 2015 and Jain *et al.* 2010).

A.2.2 **State-of-the-art**

Several studies have been conducted worldwide to set new guidelines or modify/update existing guidelines for seismic screening and evaluation of *buildings* located in specific regions. This sub-section presents some of these studies.

Ventura et al. (2005) developed a series of tools for estimating the regional seismic performance of *buildings* located in southwestern British Columbia. Based on the lateral load bearing system, construction materials, *building* height, use and age, the *buildings* were divided into 31 types. For each *building type*, a *damage* probability matrix (DPM) was established to describe the expected *damage* level after an earthquake event of certain intensity. Furthermore, for each intensity level, a probability distribution function was fit to the discrete probability values.

Grant et al. (2007) developed a three-level risk assessment procedure for prioritizing seismic intervention in school *buildings* in Italy. It is a three-level risk assessment procedure. The first ranking is based on the difference between the design ground motion and the required degree of *building* seismic resistance. The second ranking consists of a vulnerability index, which is available for a large Italian *building* stock. In the third ranking, a more accurate assessment of *seismic risk* is obtained for a simplified displacement-based methodology. The capacity ratio and the risk ranking obtained by the final assessment are used to assign priorities for seismic intervention and detailed evaluation.

Gueguen et al. (2007) developed a light vulnerability method (VULERALP) to assess the *seismic risk* of *existing buildings* in France. According to this study, the vulnerability assessment methods applied in high seismic hazard countries like Italy shouldn't be adopted in low to moderate seismic hazard countries like France. Although it is mainly based on the Italian method GNDT, VULERALP has a simpler approach in terms of rapid visual screening (RVS) and the number of structural parameters used. When tested in Grenoble-France, the VULERALP assessment results showed that this method can be applied at low cost to predict seismic vulnerability of *existing buildings*.

Karbassi and Nollet (2008) proposed a scoring procedure for seismic vulnerability evaluation for an existing group of *buildings* in the province of Quebec. This work is an attempt to update the existing 1993 NRC screening manual. The spectral acceleration values in the NBC 2005, along with HAZUS fragility and capacity curves for various *model building types* (FEMA 2003a; 2003b), were used in this method to obtain the structural vulnerability indices (*SVIs*), as well as *building* height, *irregularity*, year of design and construction, and *Site Class* modifiers. Karbassi and Nollet identified the necessity to develop structural vulnerability indices and index modifiers specific to location, rather than to directly apply the rapid visual screening (RVS) technique established in FEMA 154 (2002), which is mainly based on U.S. *seismicity*. The reference *Site Class* assumed in the calculation is *Site class C* and structural vulnerability indices are obtained for the three defined *seismicity* levels based on FEMA 310 criteria.

The assessment also shows that the impact of site amplification is higher on the proposed SVI than indicated in the 1993 NRC screening manual. Since seismic vulnerabilities may relate to soil condition rather than structural deficiencies, the proposed procedure can better identify hazardous *buildings*.

Pina (2010) presented a *seismic risk* assessment methodology and risk reduction for schools in British Columbia. The schools are ranked according to risk level, and then the seismic capacity of school *buildings* is established based on the information provided. The results of the probabilistic seismic hazard analyses are combined with results from the incremental nonlinear dynamic analyses of different structural systems, considering three types of earthquake. The *seismic risk* is computed for a set of thirty ground motions. This results in a large database of structural system responses for various soil conditions. The calculation considered *Site Class* C as the reference *Site Class*, but a simplified procedure was developed to convert structural performance of *buildings* located on *Site Class* C to the corresponding performance of school *buildings* on *Site Class D*. For a given *Site Class*, *building type*, and seismic region, this method can identify whether a retrofit is necessary.

Jain et al. (2010) proposed an RVS method for seismic evaluation of reinforced concrete (RC) *buildings* in India based on the *damage* data obtained from investigating 270 RC-frame *buildings* in Ahmedabad after the 2001 Bhuj Earthquake. The proposed method consists of six vulnerability parameters, including: the presence of a basement, number of *storeys*, apparent quality, re-entrant corners, open *storeys* and short columns. A performance score is predicted from the *seismic zone* and soil condition.

Tischer et al. (2012) proposed a seismic screening system that is specifically developed for school *buildings* in Quebec to address limitations in the applicability of existing screening procedures and guidelines such as the 1993 NRC screening manual and FEMA 154, which were developed for general *building* stock without considering school *buildings* with special characteristics. The 1993 NRC screening manual does not comply with the most recent NBC (i.e. NBC 2010). In addition, FEMA 154 was developed for the U.S. Based on the extensive characterization of 101 public school *buildings* in the city of Montreal, Tischer et al. (2012) confirmed that schools are classified as low-rise structures that use a limited number of lateral load resisting system types (LLRS).

The new proposed method is a score procedure based on the FEMA 154 methodology and capacity spectrum method. During the visual screening of school *buildings*, a data collection form is completed. Whenever possible, *building* plans and other documentations are used for additional details. One of the three *seismicity* severity levels (Low, Medium, or High) is selected, then the LLRS is identified based on the 15 predefined *building type*s. Based on the *seismicity* level, an initial structural hazard score is determined. The final structural score is computed by adding or subtracting score modifiers to/from the initial score. These modifier values mainly depend on specific *building* characteristics. The final score estimates the probability of *building* collapse. For example, a better seismic performance is expected for *buildings* with higher scores.

Achs and Adam (2012) developed an RVS methodology for comprehensive seismic evaluation of Viennese historic masonry *buildings*. Several parameters were considered in order to predict possible *damage* and to describe and classify *building* performance under seismic action and four vulnerability classes. In this study a large-scale investigation was carried out to identify the seismic vulnerability of 375 historic brick-masonry *buildings*. From the results of this investigation, a local seismic *building* vulnerability map was established. This map provides useful information for emergency and evacuation planning and also can be used to specify *buildings* that require further investigation.

Saatcigolu et al. (2013) developed software for seismic screening of *existing buildings* located in different municipalities in Canada. The software is based on the 1993 NRC screening manual. *Seismicity*, *Site Class*, and ductility factors specified in the NBC since 2005, along with the new ductility and over-strength factors used in the NBC 2010, are incorporated in the new software. Reference hazard values are derived from the uniform hazard spectrum values of *seismicity* for each Canadian city with an earthquake return period of 2475 years. The relative soil amplification values between the new acceleration-based and velocity-based soil amplification factors and the foundation factors of older codes are established. Various structural types with different toughness and energy dissipation capacities are identified by introducing the ductility and over-strength factors in the NBC 2010.

Nanda and Majhi (2014) overviewed and compared RVS methods available worldwide, in order to propose a suitable RVS procedure for developing countries. Nanda and Majhi concluded the following:

- A more generalized approach for RVS is provided by the second edition of FEMA 154 and 2006 NZSEE guidelines.
- The NRC 1993 screening manual is related to the seismic requirements of the National *Building* Code of Canada only.
- Both Japanese and Italian methods use few *building* characteristics; therefore, ranking *buildings* using these two methods lacks clarity.
- Evaluating *buildings* using the Indian procedure is very similar to the Tier 1 method in FEMA 310.
- *Pounding* effects are not considered in Eurocode 8, which requires explicit guidance for the design professional.

- Most of the seismic deficit characteristics are not considered in the seismic assessment manual in the Balkan region.

According to Nanda and Majhi (2004), the second edition of FEMA 154 and 2006 NZSEE guidelines can be suitably combined to develop a generalized procedure for seismic evaluation of *buildings* in developing countries.

Perrone et al. (2015) proposed an RVS method for RC hospital *buildings* in Italy. The main parameters that affect structure vulnerability are identified through a sidewalk survey and a risk index. Both structural vulnerability and non-structural component vulnerability are considered in this method. The proposed RVS procedure was applied on two Italian hospitals built at the same time, in two different *seismic zones*. A safety index computed by a push-over analysis was compared with the results determined from a simplified verification method. The new RVS method was also applied to two hospitals that were *damage*d by past earthquake events. The results obtained by the push-over analysis, *damage*d hospitals and new RVS method were comparable.

Albyrak et al. (2015) proposed a methodology for *seismic risk* assessment in urban *building* stock located in a seismically active zone in Turkey. The age of *building*, number of *storeys*, existence of soft *storey*, short column, heavy overhangs, *pounding* effect, topographic effects, quality of construction and seismic zoning are the *building* criteria considered for risk assessment. Seismic screening started with a sidewalk survey around the buildings, followed by the earthquake performance score (EPS) calculation for each *building* to identify *buildings* at risk. Based on the obtained EPS values, the seismic vulnerability of *buildings* is classified into three categories: 1) high risk, 2) moderate risk, and 3) low risk. The screening results conducted on 1643 *buildings* in the city of Eskisehir, Turkey, using the proposed method, revealed that 218 *buildings* needed detailed evaluation due to their high *seismic risk.*

A.2.3 Key factors considered in seismic risk screening of existing buildings

There are major parameters known to have substantial effects on the seismic performance of *buildings*. Therefore, these parameters are essential in obtaining the *building's* final score when the seismic screening procedure is performed. This subsection overviews and discusses the above-mentioned parameters and how they are used by the different available methods for seismic screening.

A.2.3.1 Seismicity

Seismicity refers to the occurrence or frequency of earthquakes in an area of interest. According to the NBC 1990, the minimum lateral seismic base shear force V for a *building* is given by:

$$V = \frac{V_e}{R} U \tag{A.10}$$

where U is the calibration factor determined to provide the same general level of safety (component size) in the NBC 1990 as in the NBC 1985, and is equal to 0.6. R is the force modification factor reflecting the structure's inelastic energy dissipation capacity, commonly known as the "ductility-related force modification factor," and ranges from 1 for brittle structures such as unreinforced masonry to 4 for ductile redundant structures such as ductile moment resisting frames. V_e is the elastic base shear design force, calculated as:

$$V_e = \upsilon SIFW \tag{A.11}$$

where υ is the zonal velocity ratio specified in the climatic data for each seismic region provided in the NBC 1990 reflecting the intensity of design earthquake when multiplied by the seismic response factor S, which provides the design response spectrum for a unit value of υ. I is the *building's* importance factor. F is the foundation factor reflecting possible soil amplification relative to the reference soil condition, which is taken as rock or dense and/or stiff soil in the NBC 1990. W is the weight associated with tributary mass for inertia forces (i.e., dead load plus 25% of snow load, plus 60% of storage load, if present, and full content of any tank).

The effect of *seismicity* is captured by the *seismicity* factor *A*, as specified in the 1993 NRC screening manual, which is defined by the *building* location and applicable NBC edition as given in Table A.2. The *seismicity* of a location depends on the effective *seismic zone*. The effective *seismic zone* is equal to Z_v when $Z_a \leq Z_v$, or Z_v+1 when $Z_a > Z_v$, where Z_a and Z_v are the zonal acceleration and velocity, respectively, for a specific location in Canada. The *seismicity* factor *A* value ranges between 1.0 and 4.0.

Table A.2: Effect of *seismicity* (from 1993 NRC screening manual)

A	Seismicity	Design NBC	Effective *seismic zone* (Z_v or Z_v + 1 if $Z_a > Z_v$)					A =
			2	3	4	5	6	
		Pre – 65	1.0	1.5	2.0	3.0	4.0	
		65 – 84	1.0	1.0	1.3	1.5	2.0	
		Post – 85	1.0	1.0	1.0	1.0	1.0	

Since 1990, the hazard values have been revised by Geological Survey of Canada (GSC), based on the increased volume of seismic records and improved understanding of *seismicity* in Canada. The new information led to the development of new seismic hazard values, which were adopted in the NBC 2005, and subsequently in the NBC 2010, as well as the NBC 2015. The new seismic hazard values are different both in substance and manner in which it is implemented. Accordingly, the seismic hazard is based on a 2% probability of exceedance in 50 years, and is expressed in terms of uniform hazard spectra (UHS) specified for different municipalities in Canada, rather than *seismic zones* as specified in FEMA 154 and in the 1993 NRC screening manual.

In the second edition of FEMA 154, the U.S. *seismicity* is divided into three *seismicity* regions (Low, Moderate, and High). To provide further scoring accuracy, a total of five *seismicity* regions are used in the third edition of FEMA P-154, i.e. Low, Moderate, Moderately High, High, and Very High.

A.2.3.2 Soil type/conditions

The Soil Type (equivalent to *Site Class* in the NBC) has a major effect on amplitude and duration of shaking, and consequently on structural *damage*. In general, a longer duration and more damage is expected for sites with deeper and softer soils.

In the first edition of FEMA 154, the soil effect is based on the UBC and NEHRP classification of "standard" soil profiles SL1, SL2, SL3, where SL1 is rock or stable soil deposits of sands, gravels or stiff clays less than 60 m (200 feet) deep; SL2 is deep cohesionless or stiff clay greater than 60 m (200 feet) deep; and SL3 is soft to medium stiff clays or sands greater than 9 m (30 feet) deep. To account for resonance-type effects for 8- to 20-*storey buildings* sitting on SL3 type soil, an additional soil type of "SL3 & 8-20 *storeys*" is considered. For any soil type other than SL1, a negative modification factor (varying from -0.8 to -0.3) is considered.

In the second edition of FEMA 154, Soil Types A (hard rock); B (average rock), C (dense soil), D (stiff soil), E (soft soil) and F (poor soil) are introduced instead of SL1, SL2, and SL3. Finally, in the third edition of FEMA P-154, score modifiers are provided for Soil Type A or B and for Soil type E. Basic scores were calculated assuming Soil Type CD (the average of Soil Types C and D). Therefore, no score modifier applies when one of these two Soil Types occurs. There is no score modifier for Soil Type F because *buildings* on Soil Type F cannot be screened using the RVS procedure. If a *building* is on Soil Type F, it is triggered for detailed seismic evaluation.

The effect of soil condition is considered with the soil condition factor (B), as specified in the NRC screening manual, which is determined by the type of dominant soil under the *building* and the applicable NBC edition, as given in Table A.3. There are five different soil categories considered in the NRC screening manual, namely rock or stiff soil less than 50 m deep, stiff soil greater than 50 m deep, soft soil greater than 15 m deep, very soft or liquefiable soils, and unknown soil condition. The value of soil condition factor *B* can vary between 1.0 and 4.0. These factors are based on the design practice at the time, and do not consider the differences between short and long period *building* responses and the influence of shaking intensity.

Table A.3: Effect of soil condition (from the 1993 NRC screening manual)

B	Soil conditions	Design NBC	Soil category					B =
			Rock or Stiff Soil	Stiff Soil > 50 m	Soft soil > 15 m	Very Soft or Liquefiable soil	Unknown Soil	
		Pre – 65	1.0	1.5	2.0	3.0	4.0	
		Post-65	1.0	1.0	1.3	1.5	2.0	

In the 1994 edition of NEHRP Recommended Provisions for Seismic Regulations for New Buildings, a new soil classification with period-dependent site coefficients was introduced in the

U.S. Six Soil Types were defined from type A to F, ranging from hard rock (Soil Type A) to poor soil (Soil Type F). This site classification was adopted by the NBC 2005 and its later editions. For the U.S., the reference Soil is Type B, while the reference *Site Class* in Canada is defined as *Site Class* C.

Soil type/condition is one of the main parameters used to calculate the structural scoring according to the Indian and Italian seismic screening procedures.

A.2.3.3 Building type

The type-of-structure effect is taken into consideration in type-of-structure factor *C*, as specified in the 1993 NRC screening manual, and is determined by the *building's* type of structural system (15 specified types) and the applicable NBC edition as given in Table A.4. Both *building* materials (i.e., wood, steel, concrete, precast, masonry infill and masonry) and structural system (moment frame, shear wall, etc.) are considered in the 1993 NRC screening manual (Table A.5). The value of type of structure factor varies between 1.0 and 3.5. A low value indicates that the structure has sufficient resistance and deformability to absorb induced seismic energy, whereas a high value indicates relatively brittle behaviour or limited deformability. The values for C were related to the ratio between the parameter K (numerical coefficient for structural behaviour) in the earlier editions of the NBC and the inverse of R (force modification factor) in the NBC 1990.

Table A.4: Type-of-structure factor (from the 1993 NRC screening manual)

C	Type of structure	Design NBC	Construction type and symbol (see p. 1)													A =
			Wood		Steel				Concrete		Precast		Masonry infill	Masonry		
			WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW, CIW	RML, RMC	URM	
	(BM = Benchmark Year, see p.1)	Pre – 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5	3.5	
		70 – BM	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5	
		Post – BM	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	

Table A.5: Type of structure (from the 1993 NRC screening manual)

Dominant material	Description of type of structure	Symbol
WOOD	Wood, light frame	WLF
	Wood, post and beam	WPB
STEEL	Steel moment frame	SMF
	Steel braced frame	SBF
	Steel light frame	SLF
	Steel frame with concrete shear walls	SCW
	Steel frame with infill masonry shear walls	SIW
CONCRETE	Concrete moment frame	CMF
	Concrete shear walls	CSW
	Concrete frame with infill masonry shear walls	CIW
	Precast concrete frame	PCF
	Precast concrete wall	PCW
MASONRY	RM bearing walls with wood or metal deck diaphragms	RML
	RM bearing walls with concrete diaphragms	RMC
	Unreinforced masonry bearing wall *buildings*	URM

Note: RM = Reinforced masonry

In the first edition of FEMA 154 there were 12 *building type*s (compared to 15 types in the 1993 NRC screening manual). In the second edition of FEMA 154, the number of *building type*s increased to 15, thus matching the number of *building type*s in the 1993 NRC screening manual, yet with minor differences in the definitions for wood and precast concrete *building type*s. In the third edition of FEMA P-154, the number of *building type*s increased to 17. Two new *building type*s were added due to: 1) recognizing manufactured housing (MH) as a new *building type*, and 2) refining the wood *building* types, increasing from two to three different types.

It is also noteworthy that, in all three editions of FEMA 154, tilt-up structures are recognized as one *building type*. In the 1993 NRC screening manual, precast concrete wall (PCW) is considered similar to the PC1 (i.e., tilt-up) *building type* in FEMA 154, which includes both precast and tilt-up wall panels.

In the Indian procedure, *buildings* are classified into five types, as listed below, and given an *n*-factor. This *n*-factor accounts for *building* ductility and energy dissipation.

- Unreinforced masonry load bearing wall *buildings* (*n*=1)
- Reinforced masonry load bearing wall *buildings* with stiff diaphragms (*n*=2)
- Reinforced concrete moment frames (*n*=3)

- Reinforced concrete frames with infill masonry shear walls (n=4)
- Reinforced concrete shear wall *buildings* (n=5)

The lateral load resisting system and organization is also used as a scoring parameter in the Italian procedure, while in southwestern British Columbia, *buildings* are divided into 31 types in the tools from Ventura et al. (2005), to estimate *buildings'* regional seismic performance.

A.2.3.4 Building irregularity

Building irregularities may lead to significant deficiencies in seismic performance. Past seismic events showed that most *building* failures were attributed to structural *irregularities* (Saatcioglu et al. 2013). Soft *storeys*, short columns, vertical discontinuities or significant changes in strength or vertical structural member stiffness significantly increase seismic demands. On the other hand, plan or horizontal *irregularities* resulting from an irregular *building* shape and/or significant eccentricities between the center of mass and center of rigidity may lead to excessive torsional stresses, thus increasing seismic deformation demands.

In the first and second editions of FEMA 154, the plan *irregularities* took re-entrant corners into account (i.e., L, U, H, T, E, or other irregularly shaped *buildings*). Torsion was also considered one type of plan *irregularity*. Torsion is the eccentric stiffness in plan (e.g. corner or wedge-shaped *buildings* or any *building type* with highly unsymmetrical lateral load resisting system or concentrated at some distance from the *building's* centre of gravity). Also in the first and second editions of FEMA 154, vertical *irregularities* were considered for *buildings* with major cantilevers, elevation steps, inclined walls and discontinuity in load path, major setbacks or other structural features causing major stiffness change in the *building's* upper *storeys* or *buildings* on a hill. Soft *storey* was also considered as one type of vertical *irregularity*. Soft *storey* is the structural feature resulting in a major stiffness decrease in the lateral load resisting system at one floor (e.g. discontinuous shear walls, openings on all sides of *building*), typically at ground floor due to large openings or tall *storeys* for commercial purposes. Short column was indirectly addressed in vertical *irregularities*. Short columns are columns designed with full *storey* height, but restrained by half-walls or spandrel beams.

In the third edition of FEMA P-154, if one or more plan *irregularities* are identified in the *irregularities* section of the form, the plan *irregularity* score modifier should be selected. On the other hand, and to maintain the simplicity of Level 1 screening, FEMA P-154 divided vertical *irregularities* into two levels (severe and moderate), rather than different types. If one or more severe vertical *irregularities* are identified in the *irregularities* section of the form, the *severe vertical irregularity* score modifier should be selected. If one or more moderate vertical *irregularities* are identified, and no severe vertical *irregularities* exist, the *moderate vertical irregularity* score modifier should be selected. A Level 2 optional screening is provided to refine the Level 1 screening score, including more types of both vertical (i.e., sloping site, weak/soft *storey*, in-plane setback, out-of-plane setback, short column/pier, split level, and others) and plan *irregularities* (torsion, non-parallel system, re-entrant corner, diaphragm opening, and beams not aligh with columns).

The effect of *building irregularities* is considered with *building irregularity* factor *D*, as specified in the 1993 NRC screening manual, which is determined by seven different types of *irregularities* as given in Table A.6. Types of *irregularities* include:

- Vertical *irregularity* (sudden changes in plan dimensions over height);
- Horizontal *irregularity* (irregular *building* shapes in the horizontal plane);
- Short concrete columns restrained by walls (resulting in columns with short unsupported length);
- Soft *storey* (severe reduction of stiffness from *storey* to *storey*);
- *Pounding* (separation between *buildings*, in mm, less than 20×Zv×number of *storeys*), where Zv is the NBC velocity-related *seismic zone;*
- Major modifications (change in function and use or significant addition to the *building*);
- *Deterioration* (*damage*d or poor condition of structural elements);
- None.

Table A.6: Effect of *building irregularities* (from 1993 NRC screening manual)

D	*Building irregularities*	Design NBC	1. Vertical	2. Horiz.	3. Short concrete columns	4. Soft storey	5. pounding	6. Modifi-cation	7. Dete-rioration	8. None	D = product of circled Numbers (Max of 4.0) =
		Pre – 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3	1.0	
		Post – 70	1.3	1.5	1.5	1.5	1.3	1.0	1.3	1.0	

The 2006 NZSEE guidelines place much greater emphasis on the presence of structural *irregularities*, such as torsion and weak *storey*, for the earthquake vulnerability of *buildings* (Nanda and Majhi 2013). *Irregularity* index is also introduced in the Japanese seismic evaluation standard (JBDPA 2002). The *building's* architectural features and plan *irregularities* must be identified in the capacity index proposed by Yakut (2004).

A.2.3.5 Building importance

The effect of *building* importance (*occupancy* characteristics) is considered in the *building* importance factor *E*, as specified in the 1993 NRC screening manual. It is determined by the *building occupancy* type and density, along with the applicable NBC as given in Table A.7. The *building* importance factor considers *post-disaster buildings* and special operational requirements, and varies between 0.7 and 3.0.

Table A.7: Effect of *building* importance (from the 1993 NRC screening manual)

		Design NBC	Low Occupancy N < 10	Normal occupancy N = 10 - 300	School, or High Occupancy N = 301 - 3000	Post disaster, or Very High Occup. N > 3000	Special Operational Requirements	E =
E	*Building* Importance	Pre – 70	0.7	1.0	1.5	2.0	3.0	
		Post – 70	0.7	1.0	1.2	1.5	2.0	

N = occupied area x *occupancy* density x duration factor* = ... x . . . . x . . . . =

Primary use:	*Occupancy* density Persons/m²	Average weekly hours Of human *occupancy*
Assembly	1	5 – 50
Mercantile, Personal service	0.2	50 – 80
Offices, institutional, manufacturing	0.1	50 – 60
Residential	0.05	100
Storage	0.01 – 0.02	100

* Duration factor is equal to the average weekly hours of human *occupancy* divided by 100, not greater than 1.0

In the third edition of FEMA P-154, while it is required to identify *building occupancy* (i.e., assembly, commercial, emergency services, industrial, office, school, utility, warehouse or residential), the *building* importance effect is not considered in the scoring system. This emphasizes the fact that the *consequences of failure* are not addressed in FEMA P-154. Furthermore, the historical, government, and shelter *occupancy* classes are also identified as additional designations in the third edition of FEMA P-154.

A.2.3.6 Benchmark year

In general, *building* codes have periodic editions, which include revisions regarding seismic design/detailing requirements for new *buildings*. The code edition in which substantial improvements in seismic code requirements (e.g., ductility capacity for inelastic energy absorption and seismic detailing requirements) were adopted and enforced (which may differ for each type of structure and jurisdiction) is referred to as the benchmark year (FEMA 2015). Any *building* designed and constructed to the benchmark year or after is referred to as a *post-benchmark building*. *Post-benchmark buildings* are expected to have superior seismic performance because more seismic design/detailing requirements were adopted and enforced. An *existing building* that is qualified as a *post-benchmark building* is provided with positive *post-benchmark* score modifiers in FEMA P-154 (2015). The determination of benchmark year is a complex task and varies according to type of structure. The key factor in specifying the appropriate benchmark year for each *building type* is finding the edition of the code in which major improvements were made to its seismic design provisions.

In the 1993 NRC screening manual, a value of 1.0 is applied to type-of-structure factor C, if the *building* was designed and constructed to an applicable NBC edition (i.e. the NBC 1985 for concrete structures and the NBC 1990 for other structures) or after.

To obtain the structural vulnerability indices (SVIs) using the score procedure proposed by Karbassi and Nollet (2008), the *building's* year of design and construction must be defined so that it can be compared to the *building's benchmark NBC edition* (i.e. the NBC edition in which significantly seismic improvements were adopted and enforced). Also, in the methodology for *seismic risk* assessment proposed by Albyrak et al. (2015), *building age* is required for the same reason mentioned above.

A.2.3.7 Pounding

During an earthquake, adjacent *buildings* with insufficient separation may impact each other or pound as a result of different fundamental periods. The source of the difference in fundamental periods between adjacent *buildings* can be due to the difference in; 1) floor levels, 2) floor masses, and 3) *building* heights. *Pounding* can alter the dynamic response of both *buildings* and impart additional inertial loads on both structures (Rai 2005), thus the seismic performance of a *building* can be significantly affected. Accordingly, *buildings* subjected to *pounding* sustain more severe *damage* at higher floors. Since *pounding* is an observable condition, it should be captured during the visual screening stage.

The potential for *pounding* should be evaluated using the calculated drifts for both *buildings*. Johnson et al. (1992) and Kasai et al. (1990) proposed approximate analytical methods for assessing the effects of *pounding*, including time history analyses and elastic response spectrum analyses, respectively. However, for many *buildings* or within the capability of many design practitioners, the use of such approaches may not prove practical (NZEE 2006).

Kasai et al. (1990, 1992) and Carr and Moss (1994) concluded that column and *storey* shear forces in the taller *building* above *pounding* level can be increased by up to 100% or more. Therefore, and in order to ensure some account of *pounding* effects is made, an alternative simplified approach was proposed where a midrange increase in design shear was been adopted and sophisticated analyses were completely avoided (NZEE 2006).

Cole et al. (2012) published the report entitled, "Building Pounding State of the Art: Identifying Structures Vulnerable to Pounding Damages." This document contains a review of the available literature on *pounding*. It also presents six critical *building* configurations vulnerable to *pounding* that might affect the likelihood of structural collapse. The six configurations presented are: 1) floor-to-column, 2) adjacent *building* with greatly different mass, 3) *building* with significantly different height, 4) external (end) *buildings* in a row, 5) *building* subjected to horizontal plan torsion, and 6) *buildings* made of brittle materials. According to Cole et al. (2012), *buildings* subjected to *pounding* but without the above characteristics are less likely to experience detrimental effects. Maison et al. (2012) used computer modelling to simulate actual earthquake response and analyzed twenty-two hypothetical *pounding* situations for both as-built and retrofitted *buildings*. The result of this study revealed that, for a typical *pounding* situation, there was a 14% increase in collapse rate for corner *buildings* at design earthquake intensities.

Pounding effect is used as a score modifier (-0.5) for S1, S2, S4, C1, and PC2 *building type*s in the first edition of FEMA 154 for Low, Medium, and High *seismicity* areas. In the second edition of FEMA 154, to facilitate screening by shortening the time necessary to complete the Data Collection Form, the *pounding* effect is not addressed. In the third edition of FEMA P-154, if *pounding* potential is identified, the score is not adjusted on the Level 1 Data Collection Form, but a detailed structural evaluation is automatically triggered. *Pounding* effect can be addressed in more detail on the Level 2 Optional Data Collection Form. Three *building* configurations are deemed important to capture: floor to columns (Figure A.7), *buildings* with significantly different heights (Figure A.8), and external (end) *buildings* in a row (Figure A.9).

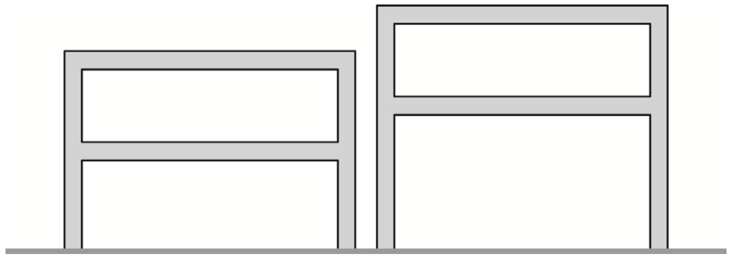

Figure A.7: Floor to column configuration (from FEMA P-154 2015)

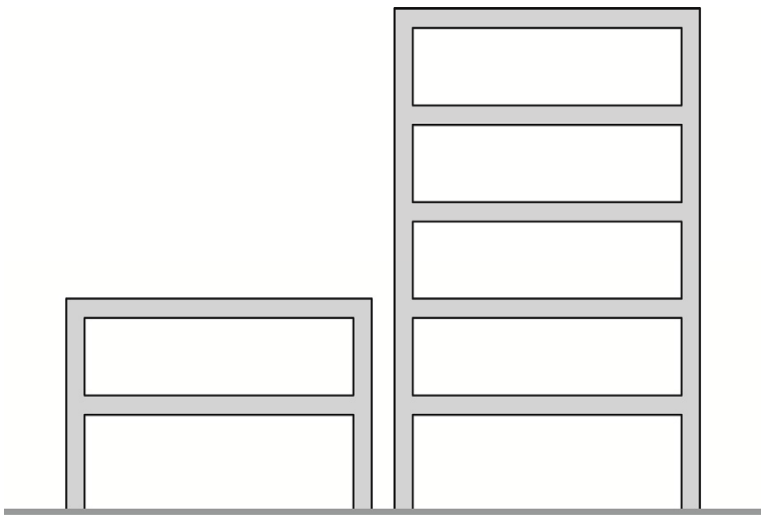

Figure A.8: *Buildings* of different heights configuration (from FEMA P-154 2015)

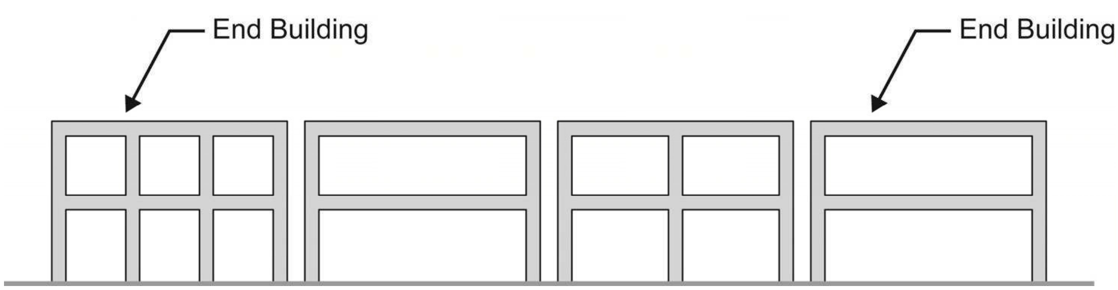

Figure A.9: End *building* configuration (from FEMA P-154 2015)

According to FEMA 310 (1998), the *building* shall not be located closer than a distance equivalent to 4% of its height to an adjacent *building*, while, according to New Zealand Code, a separation of 2% of the *storey* height induces *pounding*. *Pounding* is not addressed in the Balkan seismic assessment manual and Eurocode 8 (Rai 2005).

In the 1993 NRC screening manual, *pounding* is considered if separation between *buildings* is less than 20×Z_v×number of *storeys*. *Pounding* usually applies to frame *buildings*, but it may also apply to wood light frame *buildings*.

Pounding effect is also considered by Albyrak et al. (2015) for *seismic risk* assessment in urban *building* stock located in a seismically active zone in Turkey.

A.2.3.8 Building deterioration and damage

As part of the *seismic risk* screening, the screener needs to assess whether the *building* is constructed of sound materials and is in "good" condition. In other words, it must be verified that the level of *deterioration* and *damage* present in an *existing building* does not raise any "significant" concern. *Damage* and *deterioration* of structural elements can significantly impact an *existing building's* expected seismic performance.

In the 1993 NRC screening manual, a *deterioration* factor (*building* condition factor) is included when structural elements are *damaged*, or when the *building's* poor condition is visually apparent. Corroded reinforcement or structural steel members, rotted wood elements, and poor concrete or masonry elements are different forms of poor conditions.

While the poor condition factor is addressed in the first edition of FEMA 154, this factor does not have a score modifier in the second and third editions. However, according to the third edition of FEMA P-154, if the screener has identified any significant *damage* or *deterioration* during screening, the "significant *damage* /*deterioration* to the structural system" box should be checked on the Level 1 Data Collection Form. A detailed structural evaluation is then recommended for any *building* with significant *damage* or *deterioration*.

The *building's* apparent quality is considered for structural scoring in the seismic evaluation methods proposed by Jain et al. (2010) and Saatcioglu et al. (2007). In the Japanese seismic evaluation standard (2002), a time-dependent *deterioration* of the *building* (*T*), which is based on data collected through field inspection, is used to obtain the seismic performance index. The quality of the SFRS is also required for final scoring, according to the Italian procedure.

A.2.3.9 Remaining occupancy time

According to the ASCE/SEI 41-13 (2013), an *existing building* with shorter *remaining occupancy time* than new *buildings* has a smaller chance of experiencing a code level earthquake event over its remaining life. An *existing building* generally has a smaller chance of experiencing a code level event (smaller return period) over its remaining life span, compared to a new *building*. Knowing the risk associated with a *remaining occupancy time* of less than 50 years (i.e. typical design life for new *buildings*) facilitates the decision on whether to perform a more detailed *seismic risk* assessment for an *existing building*.

According to NIST Standards of Seismic Safety for Existing Federally Owned and Leased Buildings (2012), an *existing building* with a *remaining occupancy time* less than five (5) years is exempt from *seismic risk* assessment. To investigate the applicability of the preceding recommended five-year threshold for exempting an *existing building* from *seismic risk* assessment, one might compare the *seismic risk* in a *building* with a code level earthquake with 2% probability of exceedance in fifty (50) years to an *existing building* with an earthquake with 2% probability of exceedance in five (5) years (i.e. *remaining occupancy time*). If the expected *seismic risk* in both preceding *buildings* (i.e., in 50 years and in 5 years) is comparable, one might accept exempting an *existing building* with five years' *remaining occupancy time* from *seismic risk* assessment.

A.2.3.10 Retrofit

In FEMA P-154 (2015), a positive score modifier may be applied in Level 2 Optional Data Collection Form if the *building* has been comprehensively retrofitted (e.g., the entire seismic force-resisting system has been strengthened). This retrofit score modifier cannot be applied when elements are added to mitigate localized hazards (e.g., added wall ties or parapet bracing).

Partial retrofits and in-progress or incremental retrofits are not recognized as comprehensive retrofits, and thus do not warrant a retrofit modifier. If the retrofit appears to effectively counteract an observed deficiency, the screener can simply not apply the deficiency nor the retrofit modifier. Conversely, if a retrofit introduces a deficiency (e.g., by introducing a torsional *irregularity*), the screener should apply the appropriate score modifier for the deficiency and comment that the retrofit produced that deficiency.

It is unlikely a screener will be able to see all the elements in a lateral load path during visual screening. However, the retrofit score modifier can be applied if vertical elements in a seismic retrofit such as moment frames, braced frames, and sometimes shear walls are observed. Often, it is possible to see the vertical elements of a seismic retrofit. These vertical elements are likely part of a comprehensive retrofit when they are added. For example, it would be highly expected that ties between the diaphragm and walls have been installed, as well as parapet and gable bracing if braced frames were added to a URM *building*.

When the retrofit is already documented in available construction drawings, the applicability of the retrofit score modifier is identified at the pre-field planning stage of the seismic screening process. However, when the screener observes evidence of a retrofit in the field but the drawings are not available, the effectiveness of the retrofit may be left to the screener's judgement (FEMA P-154 2015). It is not guaranteed that the *building* will receive a score above the cut-off score (i.e 2.0) when it is retrofitted.

A.2.3.11 Redundancy

Redundancy refers to the quality of having alternative load paths in a structure by which lateral forces can be transferred, allowing the structure to remain stable following the failure of any single element. *Buildings* with sufficient *redundancy* are expected to perform better in an earthquake than *buildings* with insufficient *redundancy*, as have been observed from previous major earthquakes.

In FEMA P-154 (2015), a positive *redundancy* modifier is considered in the Level 2 Optional Data Collection Form. *Building redundancy* is considered sufficient if the *building* has at least two bays of seismic force-resisting elements on each side of the *building,* in each direction. For *buildings* with shear walls, if the number of bays is not clear, then a bay can be defined as at least the height of the *storey*.

A.3 Seismic risk screening of non-structural systems

A *building's seismic risk* depends mainly on the combined performance of structural and non-structural components. In general, performance objectives of structural components are prescribed by *building* codes and standards. On the other hand, performance objectives for non-structural components are defined by the owner and the *seismic risk* assessment team. Performance objectives for non-structural components can be selected based on the owner/operator's needs, local *building* regulations, as well as direct and indirect economic considerations. Furthermore, it is required that some non-structural components continue to function during or immediately after an earthquake to satisfy mandatory life safety requirements.

A rapid visual screening (RVS) procedure involves collecting valuable information from non-structural components. While non-structural hazards are deemed very important in high seismic regions, they are typically less important in areas of low *seismicity*. Non-structural components, however, can pose a significant life-safety threat in the event of rare large earthquakes. Passersby have been killed by dislodged heavy exterior cladding and parapets during past earthquakes. Falling non-structural ceilings, light fixtures, heavy cabinets and shelves could block exit routes, resulting in a dangerous hazard for the *building occupants*. In addition, glass shards from untempered windows and doors could injure *occupants*, particularly if located near emergency exits. Furthermore, failure of non-structural components may delay recovery of community functions.

The following is an overview of the existing major guidelines for *seismic risk* screening and evaluation of non-structural components in *buildings* in the U.S. and Canada.

A.3.1 **United States**

A.3.1.1 FEMA 154

a) FEMA P-154: Third Edition – 2015

In the third edition of FEMA P-154 (2015), the screening procedure for non-structural hazards was highly enhanced compared to the first and second editions. Chimneys, parapets, cornices, veneers, overhangs, and heavy cladding are exterior non-structural *falling hazards*. Thus, they can pose hazards to life safety if not adequately anchored to the *building*. The presence of exterior *falling hazards* may still be a danger to *building occupants* and passersby, even though the *building's* basic seismic force-resisting system may be adequate and require no further review. To help the screener identify potential hazards, several boxes in the "Exterior *falling hazards*" section were included on the Level 1 Data Collection Form.

Exterior Falling Hazards:
- ☐ Unbraced Chimneys
- ☐ Parapets
- ☐ Other: ____________
- ☐ Heavy Cladding or Heavy Veneer
- ☐ Appendages

Figure A.10: Exterior falling hazard section on the Level 1 Data Collection Form (from FEMA P-154, FEMA 2015)

Key *falling hazards* are:

- **Unbraced chimneys:** In older masonry and wood frame dwellings, unbraced, unreinforced masonry chimneys are common. Usually, these chimneys are inadequately tied to the structure and may fall as a result of moderate to strong shaking. If in doubt as to whether the chimney is braced or unbraced, it is assumed to be unbraced.

- **Parapets:** The portion of the exterior wall or façade that extends above the roof is called a parapet. Parapets are constructed of unreinforced masonry, such as brick, stone or concrete block. During an earthquake, these elements can break and fall onto the roof or out into the street. Occasionally, it is difficult to identify if a façade projects above the roofline to form a parapet, and, if there is a parapet, and it is often difficult to identify if it is braced. Satellite imagery is used in some cases to verify the presence of bracing. If in doubt as to whether an unreinforced masonry parapet is braced or unbraced, it is assumed to be unbraced.

- **Heavy cladding or heavy veneer:** During an earthquake, large heavy cladding components commonly made of precast concrete or cut stone may fall off the *building* if improperly anchored. Although these components are considered non-structural, they may substantially contribute to the *building's* mass and stiffness. Thus, major changes to the *building's* mass distribution and stiffness could occur if these panels are lost, which would result in plan *irregularities* or torsion. Glass curtain walls are a source of potential falling hazard. These components, however, are not considered as heavy cladding in the RVS procedure. Another source of falling hazard*s* is improperly anchored masonry veneers. Heavy veneers, such as full thickness bricks used as the façade material in front of wood frame construction, are more hazardous than an adhered veneer that uses partial thickness masonry units. If the heavy cladding or heavy veneer connections were designed and installed before the jurisdiction adopted seismic anchorage requirements, the concern is even greater. The date of such code adoption should be established by the supervising engineer during the planning stages of the RVS process. It should be specified that heavy cladding hazards exist if the jurisdiction has not adopted cladding ordinances or if the *building* predates adopted ordinances. Conversely, if the *building* postdates adopted ordinances, then the cladding connections may not pose a hazard because they were properly designed.

- **Appendages:** Canopies and architectural elements that add detail and decorative interest to the façade are classified as *building* appendages. They may fall off the *building* during an earthquake if improperly anchored. With larger elements, the concern is greater, since they pose a significant falling hazard risk.

- **Other:** The "Other" box should be checked and supplementary details should be given in the space next to it (and in the comments section) if the screener observed a falling hazard that does not fit into any of the above categories. For example, an elevated tank on the roof could be considered as an "Other" falling hazard if it is tall, heavy and located near the perimeter of the *building* (Figure A.11).

Figure A.11: A *building* with parapets and other *falling hazards* including a canopy over a loading deck and a water tank on the roof (from FEMA P-154 2015)

To complete the Level 1 Data Collection Form from FEMA P-154, it is recommended to take pictures of the *falling hazard(s)* for detailed reporting. The RVS authority may later develop a mitigation program using this information.

The final step of Level 1 screening is to indicate whether detailed non-structural evaluation is recommended given the following considerations:

- Yes, non-structural hazards were identified and should be evaluated. This is only selected when a non-structural hazard is detected with non-structural evaluation being recommended to decide whether the observed potential *falling hazard* is an actual threat. For example, for a *building* with heavy cladding, detailed evaluation is necessary to conclude whether the cladding is properly anchored. The heavy cladding is no longer considered as a *falling hazard* if the detailed evaluation shows that it is properly anchored.

- No, non-structural hazards exist that may require mitigation, but detailed evaluation is not necessary. This is selected if a known non-structural hazard like an unreinforced brick chimney is observed. In this case, mitigation is essential or mandatory to reduce the threat but additional evaluation is not necessary.

- No, no non-structural hazards were identified. No non-structural evaluation required if no exterior *falling hazards* are seen during the screening.

- DNK. A “do not know” option can also be selected if the screener is unable to decide whether to recommend detailed non-structural evaluation. The screener should comment on the cause of the uncertainty.

At the bottom of the Level 2 form in FEMA P-154, the screener can reply to a series of statements regarding common non-structural hazards (Figure A.12), in order to judge the *building’s* estimated non-structural seismic performance. Non-structural hazards do not affect the *building’s* final score since these hazards do not strongly affect collapse probability. In this portion of the form, there are seven statements addressing exterior *falling hazards* and two statements addressing interior *falling hazards* (addressed if access to the interior of the *building* is available). Each statement is verified by the screener in order to specify whether it is true, and relevant comments are added as the statements are reviewed.

OBSERVABLE NONSTRUCTURAL HAZARDS				
Location	**Statement** (*Check "Yes" or "No"*)	**Yes**	**No**	**Comment**
Exterior	There is an unbraced unreinforced masonry parapet or unbraced unreinforced masonry chimney.			
	There is heavy cladding or heavy veneer.			
	There is a heavy canopy over exit doors or pedestrian walkways that appears inadequately supported.			
	There is an unreinforced masonry appendage over exit doors or pedestrian walkways.			
	There is a sign posted on the building that indicates hazardous materials are present.			
	There is a taller adjacent building with an unanchored URM wall or unbraced URM parapet or chimney.			
	Other observed exterior nonstructural falling hazard:			
Interior	There are hollow clay tile or brick partitions at any stair or exit corridor.			
	Other observed interior nonstructural falling hazard:			
Estimated Nonstructural Seismic Performance (*Check appropriate box and transfer to Level 1 form conclusions*)				
	☐ Potential nonstructural hazards with significant threat to occupant life safety →Detailed Nonstructural Evaluation recommended			
	☐ Nonstructural hazards identified with significant threat to occupant life safety →But no Detailed Nonstructural Evaluation required			
	☐ Low or no nonstructural hazard threat to occupant life safety → No Detailed Nonstructural Evaluation required			

Figure A.12: Portion of Level 2 Form for non-structural hazards (from FEMA P-154 2015)

The Level 1 and 2 statements reflect similar *falling hazards,* but Level 2 statements are more specific. Exterior and interior non-structural hazards described on the Level 2 form are listed below:

Exterior

- There is an unbraced, unreinforced masonry parapet, or unbraced, unreinforced masonry chimney.
- There is heavy cladding or heavy veneer.
- There is a heavy canopy over exit doors or pedestrian walkways that appear inadequately supported.
- There is an unreinforced masonry appendage over exit doors or pedestrian walkways.
- There is a sign posted on the *building* that indicates *hazardous materials* are present.
- There is a taller adjacent *building* with an unanchored URM wall or unbraced URM parapet or chimney.
- Other observed exterior non-structural falling hazard.

Interior

- There are hollow clay tiles or brick partitions at any stair or exit corridor.
- Other observed interior non-structural falling hazard.

After carefully reviewing the statements, the *building's* non-structural seismic performance is predicted based on the screener's judgment. One of the three statements described below can be selected.

- **Potential non-structural hazards with significant threat to occupant *life safety*:** Detailed non-structural evaluation recommended. This statement is selected when a potential non-structural hazard is identified and additional evaluation may determine that the observed hazard is not an actual threat to occupant *life safety*. For example, detailed evaluation of a heavy cladding on a *building* may reveal that it is not a threat because it is properly anchored.

- **Non-structural hazard identified with significant threat to occupant *life safety*:** No detailed non-structural evaluation required. This statement is selected when a non-structural hazard is identified and no additional evaluation is required. For example, additional evaluation of an unbraced, unreinforced masonry chimney will not be helpful to prove that the chimney is not a non-structural threat.

- **Low or no non-structural hazard threat to occupant *life safety*:** No detailed non-structural evaluation required. This statement is selected only when all of the above statements are false. However, when one of the statements is true but the screener is not certain that the observed hazard is a threat to *life safety*, the screener should check this box and explain why the *falling hazard* is not a threat in the comments section.

b) FEMA 154: Second Edition - 2002

Non-structural *falling hazards* such as chimneys, parapets, cornices, veneers, overhangs and heavy cladding are also considered in the RVS procedure described in the second edition of FEMA 154. A series of four boxes are included in the Data Collection Form as shown in Figure A.13 to indicate the presence of non-structural *falling hazards*. The *falling hazards* of major concern are: 1) unreinforced chimneys, 2) parapets, and 3) heavy cladding.

The appropriate box is checked if any of the above non-structural *falling hazards* is observed. The "Other" box should be checked if there is any other hazard and the type of hazard is described in the line beneath this box. The comments section can be used if additional space is required. This information can be used later by the RVS authority as a basis for notifying the owner of potential problems.

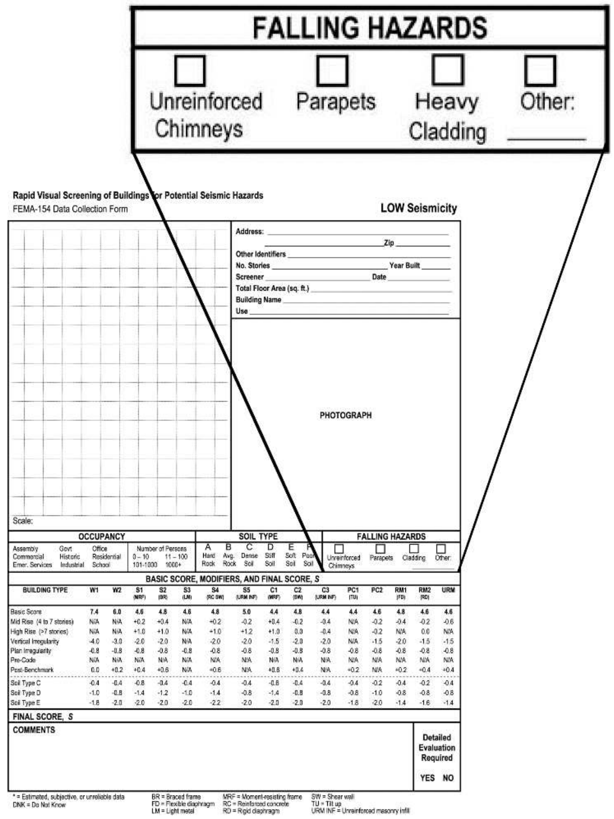

Figure A.13: Section of the Data Collection Form for documenting non-structural hazards (from FEMA 154 2002)

A.3.1.2 <u>Fourth edition of FEMA E-74</u>

The fourth edition of FEMA E-74, "Reducing the Risks of Non-Structural Earthquake Damage – A Practical Guide" (FEMA 2012) is recommended if the results of the rapid visual screening indicated that non-structural evaluation is necessary.

Although E-74 is used for detailed evaluation, it's a first step survey toward reducing the non-structural hazards in an *existing building*, and it could be useful in new screening techniques. This step consists in performing a survey to assess the extent and magnitude of potential risks. It contains guidelines for non-structural components and describes the inventory form, checklist, and risk ratings. In order to make informed decisions regarding non-structural *seismic risk*s, owners and managers will need to address the following:

- Type identification of non-structural components that are present in a particular *building*.
- Adequacy verification of bracing and anchoring of non-structural components.
- Prediction of earthquake performance of a specific non-structural item, and consequences of failure of that item in terms of life safety, property loss, and functional loss.

- Key problem to deal with in case the decision is made to upgrade the facility.

Identifying non-structural components that may be vulnerable to earthquake *damage* is the goal of a facility survey. The following basic questions should be kept in mind during the survey:

- Is there a possibility that anyone could be hurt by the item considered during an earthquake? (*Life Safety*)
- Is there a possibility that large property loss occurs? (Property loss)
- Could a serious problem occur as a result of interruptions and outages? (Functional Loss)

For some components, failure may result in both direct and indirect *damage*. It is also important to consider the consequences of failure of each item, and not only view each item as a discrete object that could tip or fall and hurt someone. For example, gas-fired residential water heaters have rarely injured anyone as they fall, but they have frequently caused post-earthquake fires due to ruptured gas lines.

The survey ranks the different hazards of non-structural components. The ranking is based on the four factors described below.

1. **Shaking intensity**: The seismic map presented in Figure A.14 can be used for particular geographic locations in the U.S. to estimate shaking intensity. The seismic map shows the areas that are likely to experience minimal, low, moderate, or high ground shaking during future probable maximum considered earthquake events that could affect the areas. Based on this map, the shaking intensity estimates should be adequate for items situated at or near the ground in simple, non-essential facilities. It is advisable to choose the next higher shaking intensity or to seek the advice of professional consultants for other situations. A comprehensive upgrade of non-structural components may not be warranted for areas with light shaking, unless an owner is particularly risk averse; the current code would not require many of the protective measures recommended herein, even for new construction.

2. ***Life safety* (LS) risk**: Risk of death or direct injury that requires hospitalization does not include the overall impact on a *building's* life safety systems, like losing emergency power in a hospital or losing fire detection or suppression capability. These disruptions of service are covered under "Functional loss" below.

3. **Property loss (PL) risk**: Risk of incurring a repair or replacement cost because of *damage* to the item. For example, property loss includes the cost of fixing a broken pipe but not the indirect cost of *damage* from water leakage.

4. **Functional loss (FL) risk**: Risk that the item will not function because it has been *damaged.* It considers the loss impact of the function of a component on the operation a *building,* without

considering the off-site functional impacts, such as the loss of function of a piece of equipment because of a city-wide power outage.

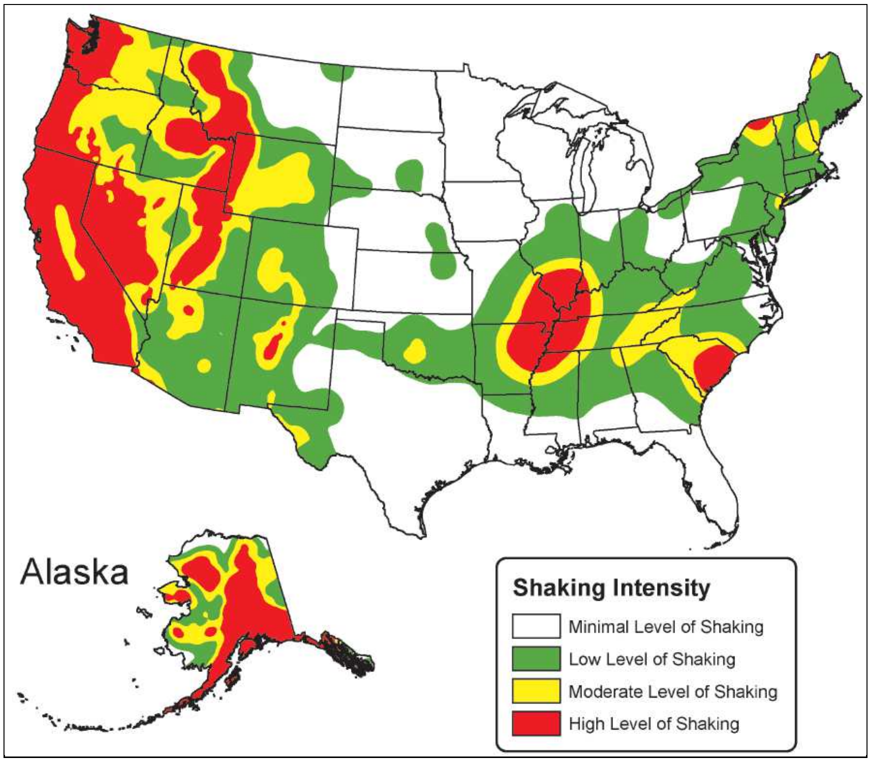

Figure A.14: Example of shaking map (from FEMA E-74)

The field survey may be done using specific forms and checklists included in Appendices C, D, and E of the FEMA E-74 document, if the shaking intensity map review indicated that non-structural hazard mitigation is a concern for the facility in question.

The blank non-structural inventory form in Appendix C is used to report field observations. To identify the facility, this form should be filled in at the start of the survey. While walking through the facility and reviewing the questions in the checklist found in Appendix D, field observations are recorded in a specific place on the inventory form. The non-compliant item from the checklist is also entered as a line item on the inventory form. Risk ratings from the checklist found in Appendix E can be added to the inventory form during the field survey or at a later time. The type of the problem observed can be added in the space provided for notes.

It is helpful to create a large list of items during the initial survey and this list may be shortened later by dropping low-priority items. It is also recommended to overestimate vulnerability and be conservative at the initial stage. The checklist of non-structural earthquake hazards shown in Figure A.15 and included in Appendix D of the FEMA E-74 document contains questions that are designed to help identify vulnerable non-structural items and potential hazards associated with each item. This checklist is filled in during the field survey. The "Noncompliance (NC)" answer on the checklist is indicative of a potential problem. In the inventory form mentioned earlier, each non-structural component of a potential problem is listed as a line item.

6.3	Architectural Components						
Item No.	**Component Name**	**Principal Concerns**	**Example**	**C**	**NC**	**NA**	**Checklist Questions (Yes=Compliance; No or Unknown=Noncompliance; NA=Not Applicable)**
6.3.1	**Exterior Wall Components**						[Exterior falling hazards are a primary concern, especially items situated above 10 feet and items that may fall over exits, walkways, or sidewalks.]
	Adhered veneer	Falling hazard	6.3.1.1				Is the adhered veneer adequately attached to the structure? [This includes relatively thin sections of tile, masonry, stone, terra cotta, ceramic tile, glass mosaic units, stucco, or similar materials attached to a structural wall or framework by means of an adhesive].
							Based on visual observations and/or tapping, is the veneer free of cracked or loose sections that may fall during an earthquake?
	Anchored veneer	Falling hazard	6.3.1.2				Is the anchored veneer adequately attached to the structure? [This includes thicker masonry, stone, or stone slab units that are attached to the structure by mechanical anchors].
							Is the masonry or other veneer supported by shelf angles or other elements at each floor?
							Is the masonry or other veneer connected to a structural back-up wall at adequate spacing?
							Has the veneer been adequately maintained? Are the anchors in good condition, free of significant corrosion, and inspected regularly?
	Prefabricated panels	Falling hazard, damage to panels and connections, broken glass	6.3.1.3				Were the panels and connections designed by an architect or engineer to accommodate the expected seismic distortion of the surrounding structure?
							Are prefabricated cladding panels detailed to allow relative movement between the panel and the structure?
							Are prefabricated panels supported for vertical loads with at least two connections per panel?

Figure A.15: Sample of non-structural earthquake hazards (from FEMA E-74)

Finally, the non-structural seismic risk ratings checklist shown in Figure A.16 and included in Appendix E of the FEMA E-74 document summarizes estimated risk ratings stated as low, medium, and high for many components, based on the component’s exposure to the levels of shaking intensity defined on the shaking map. According to FEMA E-74, risk ratings are based on the risk to life safety, property loss and functional loss for unanchored or unbraced items located at or near the base of a low-rise *building* with ordinary *occupancy*.

Example No.	Example Name	Shaking Intensity	Life Safety (LS)	Property Loss (PL)	Functional Loss (FL)	Type of Detail
	ceiling	Mod	L	M	M	
		High	M	H	H	
6.3.4.2	Directly applied to structure	Low	L	L	L	NE
		Mod	M	M	M	
		High	M	H	H	
N/A	Soffits (stucco, gypsum board, plaster)	Low	L	L	L	
		Mod	M	M	M	
		High	H	H	H	
6.3.4.3	Suspended heavy ceilings	Low	L	L	L	PR
		Mod	M	M	M	
		High	H	H	H	
6.3.5	**Parapets, Appendages, Roof Tiles**					
6.3.5.1	Unreinforced masonry parapet	Low	M	M	L	ER
		Mod	H	H	L	
		High	H	H	L	
N/A	Parapets, cornices, decoration	Low	M	M	L	
		Mod	H	H	L	
		High	H	H	L	
N/A	Hanging appendages	Low	L	L	L	
		Mod	H	H	L	
		High	H	H	M	
N/A	Clay roof tiles[1]	Low	L	L	L	
		Mod	L	M	L	
		High	M	H	M	
6.3.6	**Canopies, Marquees, Signs**					
6.3.6.1	Canopies, Marquees, Signs	Low	L	L	L	ER
		Mod	H	H	L	
		High	H	H	M	
N/A	Heavy signs or exterior billboards	Low	L	L	L	
		Mod	H	H	L	
		High	H	H	L	
N/A	Flagpoles	Low	L	L	L	
		Mod	L	L	L	
		High	M	L	L	

Figure A.16: Sample risk ratings form (from FEMA E-74)

A.3.2 **Canada**

There are two existing approaches in Canada for the *seismic risk* screening of non-structural components: the 1993 NRC screening manual (NRC 1993a) and CSA S832-14 (CSA 2014), Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings. Although these methodologies have served their purpose for *seismic risk* screening of non-structural components, an updated methodology is required to reflect the changes in the NBC 2015 (NRC 2015) and also to provide a scoring system that is consistent with structural scoring in the *Level 2 – SQST*.

A.3.2.1 <u>1993 NRC screening manual</u>

According to the 1993 NRC screening manual, the *building* components that are commonly designed by the architect or the mechanical and electrical engineers are referred to as non-structural components. Non-structural *building* components may be classified into two classes: 1) exterior non-structural components and 2) interior non-structural components.

1) Exterior non-structural components

- **Exterior cladding or veneer and window glass:** These non-structural components can fall to storefronts, streets, sidewalks, and adjacent properties if their connections to the building structure are lacking sufficient strength or ductility.

- **Parapets, cornices, ornamentations, and other appendages:** These components are very vulnerable during seismic events. Deaths of passing pedestrians caused by falling masonry parapets were repeatedly reported in past earthquakes. These non-structural type architectural features keep falling during an earthquake because they receive substantial motion amplification.

2) Interior non-structural components

- **Non-structural partition walls:** Partitions made of masonry and tiles, studs and gypsum board or plaster, demountable partitions of metal, wood or glass can be destroyed, overturned, cracked or separated from the rest of partitions. Yet, the only life-threatening partitions considered are the unsupported heavy partitions. Partitions that are adjacent to *building* entrances and exits must also be considered since their failure during an earthquake may prevent escape from the *building*.

- **Mechanical and electrical equipment:** In general, mechanical and electrical equipment like pumps, fans, piping, ducts, and electrical panels perform well during earthquakes when firmly attached to walls and floors. However, it is still possible for rigid piping and their supports or hinges to fail. Mechanical and electrical equipment has also reportedly overturned. Well-attached equipment can impact other items due to the horizontal movement they experience. This displacement should not be ignored because of possible associated danger of fire. Elevator counterweights are also likely to separate from the rails. This separation may render the elevator inoperative as a result of *damage* to structural elements, cables and cabins.

Mechanical and electrical equipment failure is generally less of a threat to life; however, after the disaster, such failure can seriously impair *building* function.

- **Interior storage water tanks and pressure vessels:** Shifting of tanks and pressure vessels from their supports is likely to occur during an earthquake. Severe *damage* to the floor and other structural components could be associated with this shifting. Nevertheless, interior storage water tank and pressure vessel equipment is considered life threatening only when they contain corrosive materials or when their failure leads to the failure of other key structural components.

The 1993 NRC screening manual provides a non-structural scoring system to assess the *seismic risk* of non-structural components. It determines the *seismic risk* of non-structural components as the product of three parameters: soil (factor *B*), *building* importance (factor *E*), and non-structural hazard (factor *F*) (Table A.8), following Eq. (A.12).

$$NSI = B \cdot E \cdot F \qquad (A.12)$$

The product of the soil and *building* importance factors correspond to seismic vulnerability, while the non-structural hazard factor corresponds to two potential hazards for *building occupants*. Those hazards are: 1) falling hazard to human life, and 2) hazards to vital operation. The non-structural hazards consider the effect of *irregularities* and structural system flexibility on the non-structural components' seismic response. Heavy non-structural components that may fall onto areas of human *occupancy*, corridors, stairs, and exits are considered as *falling hazards* to life and are divided into two classes: 1) exterior hazards, and 2) interior hazards. On the other hand, seismic *damage* that affects *buildings'* continuity of operational requirements other than *post-disaster buildings* mentioned in the NBC 1990 are considered as hazards to vital operation.

Table A.8: Effect of non-structural hazards (from 1993 NRC screening manual)

		NON - STRUCTURAL HAZARDS	Description (see p. 1)		None	Yes	Yes *	$F = \max(F_1, F_2)$
F	F_1	Falling Hazards to Life		Pre - 70 NBC	1.0	3.0	6.0	
				Post - 70 NBC	1.0	2.0	3.0	
	F_2	Hazards to Vital Operations		Any Year	1.0	3.0	6.0	
	* applies only if one or more of the following descriptors on page 1 are circled: SMF, CMF, soft storey, torsion							

A.3.2.2 <u>CSA S832 (2014)</u>

In the CSA S832 standard, non-structural components are referred to as operational and functional components (OFCs). According to CSA S832, OFCs are *building* components that are directly associated with the facility's function and operation. OFCs consist of architectural components, *building* services components, and *building* contents as shown in Figure A.17.

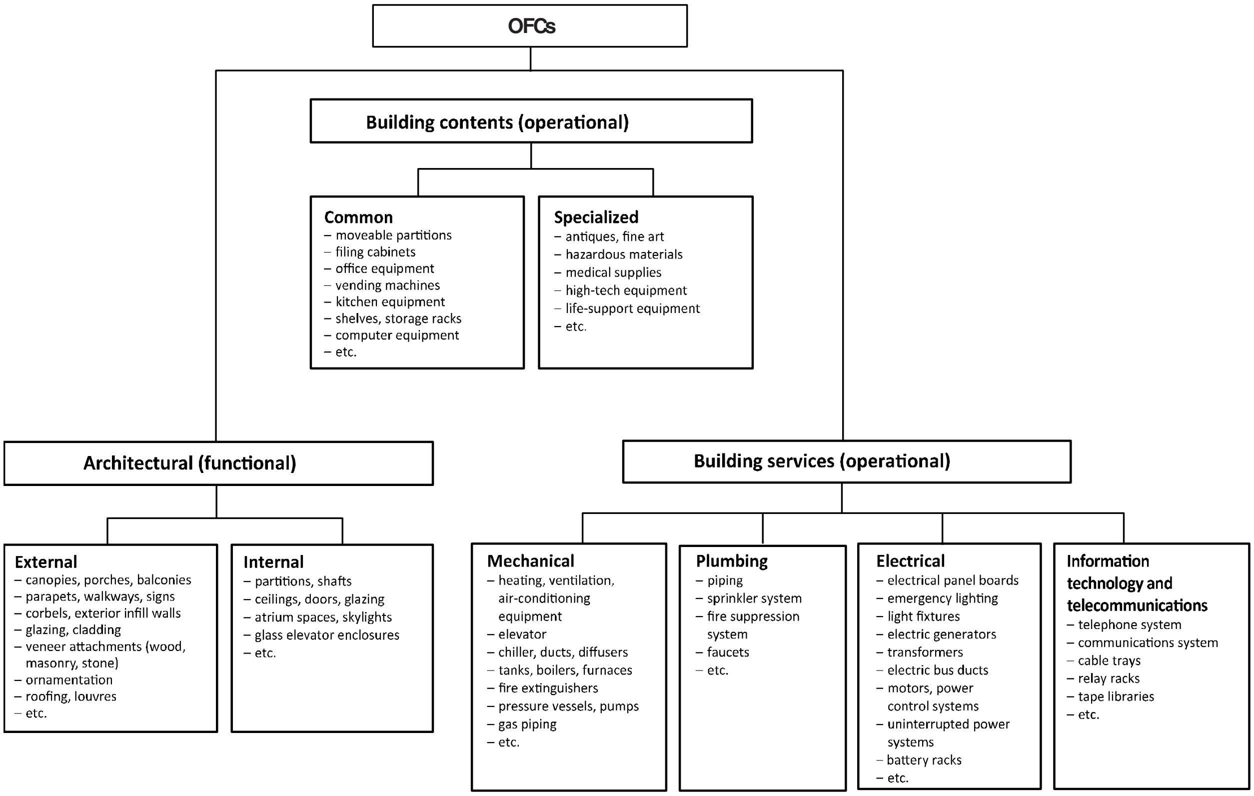

Figure A.17: Non-structural building components (from CSA S832-14).

The CSA S832-14 standard prescribes a qualitative *seismic risk* index for specific non-structural components, based on seismic parameters and requirements set out in the NBC 2010. The *seismic risk* index results from the product of the vulnerability and the consequences of individual non-structural components in accordance with Eq. (A.13). Vulnerability is based on seismic demand and parameters defining the characteristics of the non-structural components and the *building* where the components are located, while consequences are determined based on the potential number of individuals at risk, functionality and property protection.

CSA S832 defines four categories of OFC performance objectives:

1) ***Life safety (LS)***

   Preventing life-threatening movement or failure of OFCs in a *building* during an earthquake is the objective. Such movements include: falling, overturning, sliding, rolling, or swinging components that can cause serious injury; blockage of entrance/exit routes; and other effects such as fires, *hazardous material* leakage, etc.

2) ***Limited functionality (LF)***

Limited functionality performance level can be defined as the degree of *damage* to the structure and the OFCs at which the *building* is still safe for immediate austere *occupancy* or will require only minor repairs after being struck by an earthquake, in addition to the mandatory *life safety* requirements, provided that *life safety* level requirements are satisfied.

This performance objective applies to most OFCs in *buildings* in the *High importance* category (I_E = 1.3) such as schools, community centres, sports centres and other *buildings* that are intended for use as emergency shelters.

3) ***Full functionality (FF)***

Full functionality is a high-level performance objective that satisfies the requirements of the first two levels mentioned above, in addition to withstanding *damage* and remaining in service for continued use and operation after the earthquake. In general, for most OFCs and sub-systems in *Post-disaster buildings* with I_E=1.5, the post-earthquake functionality is required. This functionality is also required for all OFCs that provide life safety for *building occupants* such as emergency power systems, fire alarm and protection systems, alarm system, and firefighter elevator operation in normal importance (I_E=1) and *High importance* high-rise *buildings* (I_E=1.3).

4) ***Property protection (PP)***

This performance objective is optional and restricted to non-operational components. It depends on the type of *building*, business/*occupancy* and the owner/operator's willingness to tolerate property loss. OFCs that are assigned to meet a full functionality performance objective are *de facto* protected. Some examples of non-operational components that might require protection are antiques, fine art, material in storage, and architectural elements.

Based on a preliminary classification according to their performance objectives (LS, LF, FF, or PP), OFCs that pose no *life safety* hazards and that do not require property protection can be excluded from further risk assessment. Also, the standard excludes assessment of OFCs in *buildings* with insignificant seismic hazard. The criteria for insignificant seismic hazard is the one specified by the NBC 2010, which indicates that non-structural components with $S_a(0.2)$ of 0.12 g or less are exempted from seismic design.

The seismic risk index (*R*) can be determined for each OFC by multiplying the vulnerability index (*V*) and consequence index (*C*) as follows:

$$R = V \cdot C \tag{A.13}$$

where,

V = seismic vulnerability index, and

C = the consequence index.

Vulnerability index V can be determined based on three main characteristics: 1) ground motion characteristics, 2) *building* characteristics and 3) OFC characteristics. For each of those characteristics, an index is defined. The vulnerability index can be written as function of the three indices:

$$V = VG \cdot VB \cdot VE/10 \tag{A.14}$$

VG is the ground motion characteristics index and is expressed as the product of the spectral response acceleration value for a period of 0.2 seconds, $S_a(0.2)$, and the acceleration-based site coefficient F_a, as defined in the NBC 2010, Article 4.1.8.4:

$$VG = F_a S_a(0.2)/1.25 \tag{A.15}$$

VB is the *building* characteristics index and is based on the *building* structure's predominant type of seismic force resisting system, as defined in Table A.9.

VE is the OFC characteristics index and is obtained by the weighted sum of four rating scores:

$$VE = \sum_{i=1}^{4} RSi \cdot WFi \tag{A.16}$$

RS1: rating score for OFC restraint. The resistance to failure or *damage* of an OFC depends on the presence of lateral restraints, as well as the connections' strength and ductility.

RS2: rating score for OFC impact/*pounding*. For *buildings* that are close together or for *building* structures that contain movement joints, the adjacent structures can impact each other during earthquakes, causing OFC failure or severe *damage*.

RS3: rating score for OFC overturning. The height of the centre of gravity of an OFC resting on a horizontal surface, relative to the minimum width of its base, is critical to its overturning potential.

RS4: rating score for OFC flexibility. The level of floor acceleration response increases with height within the *building*. OFCs located on the upper levels of a *building* will experience higher acceleration than OFCs located in the lower parts of the *building*.

WFi are the *RSi* weighing factors and are represented in Table A.10.

The index *C* (in Eq. (A.13)) is defined based on assessment of the consequences of OFC failure or malfunction in relation to the OFC performance objective. It is the sum of the rating scores related to the four performance objectives. The rating score related to the LS performance objective is obtained from the *occupancy* parameter (*N*), which can be computed as follows:

N = occupied area exposed to risk × *occupancy* density × duration factor (A.17)

The rating scores related to LF and FF are defined in Table A.11.

Table A.9: Building characteristics V_B (from CSA S832-14)

	Estimated fundamental period of the building (T), s			
	$0 < T \leq 0.2$	**$0.2 < T \leq 0.5$**	**$0.5 < T$**	**Seismic force resisting system**
Number of storeys	1–2	3–4	≥ 5	Steel moment resistantframe
	1–2	3–5	≥ 6	Reinforced concrete moment resistantframe
	1–2	3–7	≥ 8	Concrete shear wall
	1	2–4	≥ 5	Braced frame
Site Class A Hard rock	1.0	1.1	1.2	
Site Class B Rock	1.0	1.2	1.3	
Site Class C Very dense soils and sof trock	1.1	1.2	1.3	
Site Class D Stiff soil	1.2	1.3	1.4	
Site Class E Soft soil	1.3	1.4	1.5	
Site Class F	1.5	1.5	1.5	

Table A.10: Determination of the seismic vulnerability index V^* for OFCs (from CSA S832-14)

Vulnerability parameters	Parameter range	Rating score (*RS*)	Weight factor (*WF*)
OFC restraint (*RS1*) (see Annex E for explanatory notes on restraint)	Full restraint	1	4
	Partial restraint or questionable restraint	5	4
	No restraint	10	4
Impact/pounding (*RS2*)	Gap adequate	1	3
Impact, pounding, and/or displacement-sensitive OFC	Gap questionable or gap inadequate	10	3
OFC overturning (*RS3*)	OFC fully restrained against overturning or $h/d \le 1/(1.2F_aS_a(0.2))$	1	2
h = distance from support or restraint to centre of gravity or top of the OFC	$h/d > 1/(1.2F_aS_a(0.2))$	10	2
d = horizontal distance between OFC supports			
F_a = acceleration-based site coefficient			
S_a = spectral response acceleration value			
OFC flexibility and locations in building (*RS4*)†	Stiff or flexible OFC on or below ground floor	1	1
	Stiff OFC above ground floor	5	1
	Flexible OFC above ground floor	10	1
Ground characteristics	$VG = F_aS_a(0.2)/1.25$	Not applicable	
VG§ = characteristic of ground motion and soil condition, expressed as the product of the spectral response acceleration value for a period of 0.2 s, $S_a(0.2)$, and the acceleration- based site coefficient, F_a, as defined in Article 4.1.8.4 of the *NBCC*			
Building characteristics	Various types of structures	See Table 4	
*VB*** is based on the predominant type of lateral-force-resisting system of the building structure			

* *Seismic vulnerability index is calculated using V = VG × VB × VE/10.*
† *Stiff OFCs shall be defined as those having a fundamental period for the OFC and its connection less than or equal to* 0.06 s. *Flexible OFCs shall be defined as those having a fundamental period for the OFC and its connection greater than* 0.06 s.
‡ *See Clause A.4.2 for an explanation of element characteristics, VE.*
§ *See Clause A.4.3 for an explanation of ground characteristics, VG.*
** *See Clause A.4.4 for an explanation of building characteristics, VB.*

Table A.11: Determination of consequence *C* for OFCs (from CSA S832-14)

Consequence parameters	**Parameter range**	**Rating score (*RS*)**
Life safety (LS) Impact on life safety from malfunction or failure of OFC during and immediately after the earthquake (e.g., items falling on or crushing people, blocking of egress, potential for f re or explosion, loss of life-support systems in hospitals, or release of toxic materials)	Threatto very few ($N \leq 1$)† Threatto few ($1 < N < 10$)† Threatto many ($N \geq 10$)†	1 5 10
Limited Functionality (LF) OFC is required for immediate austere building occupancy or occupancy with minor repairs following the earthquake	Not applicable or OFC breakdown greater than one week is tolerable OFC breakdown up to 1 week is tolerable OFC in high importance category building according to the *NBCC* ($I_E = 1.3$) and thatis not required to be fully functional OFC in post-disaster facility according to the *NBCC* ($I_E = 1.5$) and thatis not required to be fully functional	0 1 3 5
Full Functionality (F) OFC is required for post-disaster functions or for uninterrupted functionality during or immediately after the earthquake	Not applicable OFC required to be fully functional	0 10
Property protection (PP) (Optional) OFC damage can result in financial losses related to asset damage, replacement, and business interruption due to non-operational components	Score may vary from 0 to 10 as determined by the owner/operator	0–10

* *Consequence index is calculated using $C = \Sigma(RS)$, with a minimum value of 1 and a maximum value of 20.*
† *N = area × occupancy density × duration factor.*

A.3.2.3 Comparison of existing seismic risk methodologies in Canada

Table A.**12** lists the parameters affecting the *seismic risk* of non-structural components included in the 1993 NRC screening manual and CSA S832-14. In general, CSA S832-14 considers more parameters than the 1993 NRC screening manual due to the specific nature of the OFCs it considers. The 1993 NRC screening manual does not include the specific *seismicity* for the location of the *building* containing the non-structural components, the characteristics of non-structural components, ageing and *deterioration*, *pounding* and remaining *building occupancy*. The 1993 NRC screening manual does not account for ground motion characteristics of the *building* location, which results in non-structural indices independent of seismic zones. Similar to the 1993 NRC screening manual, CSA S832-14 does not assess ageing and *deterioration* and remaining *occupancy*. Furthermore, CSA S832-14 disregards the effect of year of construction and *building irregularities* on the *seismic risk* of non-structural components. To provide a more comprehensive assessment of the *seismic risk* of non-structural components, non-structural screening in the *Level 2 – SQST* considers the parameters provided in Table A.**12**. The consequences are used to determine the acceptable non-structural *seismic risk* score thresholds.

Table A.12 **Parameters affecting seismic risk**

Parameter considered in SQST	***NSI* (1993 NRC screening manual)**	***RI* (CSA S832-14)**
Seismicity (spectral response acceleration)	No	Yes
Site Class	Yes	Yes
Non-structural characteristics	No	Yes
Structural characteristics (deficiencies):		
building type	Yes[1]	Yes
irregularities	Yes[2]	No
pounding	No	Yes
ageing and *deterioration*	No	No
building design code	Yes	No
remaining occupancy	No	No
Importance and consequences	Yes	Yes

[1] Increase of the non-structural hazard factor for *building type*s SMF and CMF, soft *storey irregularity*, and torsion.
[2] Based on velocity- and acceleration-related *seismic zones*.

A.4 Ranking of a portfolio of existing buildings

A.4.1 1993 NRC screening manual

The 1993 NRC screening manual was developed to prioritize potential hazardous *buildings* for detailed seismic evaluation. The manual is qualitative in nature and based on seismic demand. A structural index (*SI*) is assigned to a *building* based on the multiplication of the following factors: *seismicity* factor A, soil condition factor B, type-of-structure factor C, *irregularity* factor D and *building* importance factor E. In addition, a non-structural index (*NSI*) is assigned to a *building* based on the multiplication of factors B, E, and an additional factor F, which is equal to the maximum of the factors for *falling hazards* to life F_1, and for hazards to vital operations F_2. *NSI* is not proportional to *SI* because the values of factor F in *NSI* and of factors A, C, and D in *SI* vary depending on specific conditions. A final score, the seismic priority index (*SPI*), is obtained from the addition of *SI* and *NSI* and used to assess the *seismic risk* for the whole *building* (see Table A.13).

Table A.13: Suggested priority thresholds of *SPI* from the 1993 NRC screening manual

Seismic priority index (*SPI*)	Priority
$SPI < 10$	Low
$10 \leq SPI \leq 20$	Medium
$20 < SPI \leq 30$	High
$SPI > 30$	Potentially hazardous

An *SI* or *NSI* of 1.0 to 2.0 would essentially indicate full compliance with the NBC 1990 and all such *buildings* could be assumed to be adequately designed for seismic safety. *Buildings* should be ranked in order of priority according to an index. The higher the index, the higher the priority. The ranking can be done in three ways: (1) according to *SPI* for the *building* as a whole, (2) according to *SI* and (3) according to *NSI*. These alternatives may be useful because non-structural hazards become more serious than structural hazards in lower *seismic zones* and it may be easy to perform detailed seismic evaluation and to upgrade non-structural components. Table A.13 provides suggested priority thresholds of *SPI* for ranking procedure 1. The *building* owner should consider the costs and benefits of seismic safety, and decide what index is an appropriate one to be used as an indicator for further detailed investigation.

A.4.2 CSA S832 (2014)

The CSA S832 provides information and methodologies to identify and evaluate seismic hazards associated with operational and functional components (OFCs), herein referred to as non-structural components, and to undertake appropriate mitigation strategies and techniques. The *seismic risk* of an individual OFC is evaluated based on a *seismic risk* index *R*, which is equal to the multiplication of the seismic vulnerability index *V* and the consequence index *C*. The CSA S832 exempts OFCs with an *R* value less than 16 from the mitigation. It also provides the suggested mitigation priority thresholds based on the *R* value. The higher the *R* value, the higher the priority (see Table A.14).

Table A.14: Suggested mitigation priority thresholds (CSA S832 2014)

Risk index (*R*)	Seismic risk level	Mitigation priority
$R \leq 16$	Negligible	Not required
$16 < R \leq 32$	Low	Low
$32 < R \leq 64$	Moderate	Medium
$64 < R \leq 128$	High	High
$R > 128$	Very High	Very High

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APPENDIX B SEISMIC RISK IN EXISTING BUILDINGS

B.1 Seismicity in Canada

Figure B.1 provides a map of Canada with seismic activity for the period of 1627-2017. Earthquake events are represented by circles increasing in size and colour darkness, in accordance with event magnitude on the Richter scale. In addition, eleven major earthquakes with magnitude greater than 6.9 are identified, based on events from 1627 to 2008, as reported by Lamontagne et al. (2008), as well as historical records available at http://www.earthquakescanada.nrcan.gc.ca. The location, date, magnitude and consequences of these major earthquakes are provided in Table B.1. The concentration of earthquakes with different magnitudes indicates that there are three major seismic zones in Canada: western, eastern, and arctic. These zones represent regions in Canada where earthquakes historically have occurred in clusters. Western and Eastern Canada are high and moderately-high seismic zones, respectively, and Arctic Canada is a moderate seismic zone. In the central part of Canada, no significant earthquake risk exists. Given the high concentration of people in Western and Eastern Canada, the western and eastern seismic zones are of particular interest for seismic screening.

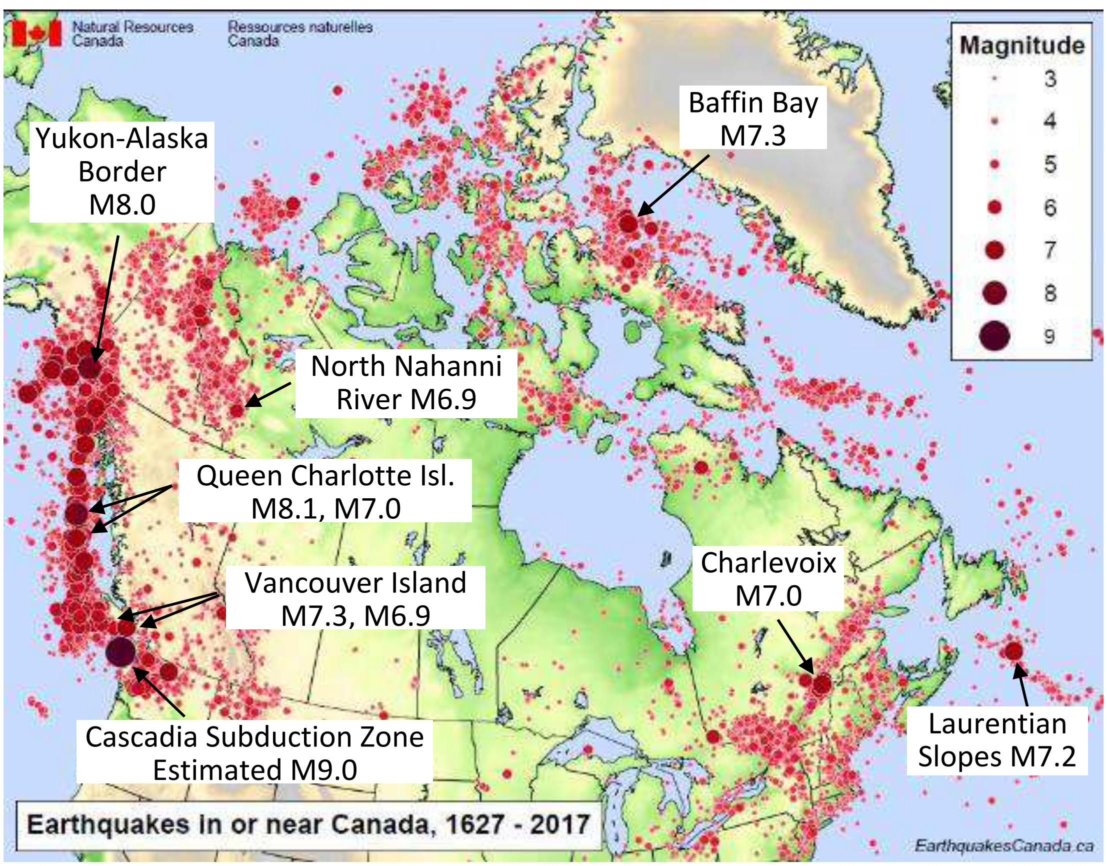

Figure B.1: Earthquakes in or near Canada from 1627 to 2017 (available online at http://www.earthquakescanada.nrcan.gc.ca/historic-historique/caneqmap-en.php)

Table B.1: Major earthquakes in Canada (data from Lamontagne et al. 2008 and http://www.earthquakescanada.nrcan.gc.ca)

Location	Date	Mag.	Landslide	Tsunami	*Damage*	Deaths
Cascadia Subduction Zone, BC	January 27 1700	9	Yes	Yes	Yes (in Japan from tsunami)	Unknown
Queen Charlotte Island, BC	August 22 1949	8.1	No	Yes	Yes	0
Yukon-Alaska Border	September 10 1899	8	Yes	Yes	Yes (in U.S.)	0
Haida Gwaii region, BC	October 27 2012	7.7	Yes	Yes	No	0
Baffin Bay, NU	November 20 1933	7.3	No	No	No	0
Vancouver Island, BC	June 23 1946	7.3	Yes	No	Yes	1
Laurentian Slope, offshore NL and NS	November 18 1929	7.2	Yes	Yes	Yes	28
Queen Charlotte Island, BC	May 26 1929	7.0	Yes	Yes	Yes	0
Vancouver Island, BC	December 6 1918	6.9	No	No	Yes	0
Charlevoix, QC	February 5 1663	7	Yes	No	Yes	0
North Nahanni River, NT	December 23 1985	6.9	Yes	No	No	0

B.1.1 **Western Canada**

The boundary between the North American and Pacific tectonic plates occurs along the west coast of the U.S., Mexico, and Canada. Subduction of the Pacific Plate under the North American Plate results in major faults that release a significant amount of energy and shake the ground. Earthquakes on the west coast of Canada occur along the faults in the offshore region, within the subducting ocean plate and continental crust. The major faults along the west coast are the Juan de Fuca subduction fault off Vancouver Island and the Aleutian Trench off the coast of Alaska. These faults define the seismic zones in the west coast and have generated earthquakes with Richter magnitudes greater than 8. There are many other fault zones throughout western Canada that are also helping to release the stress that is built up as the tectonic plates move past one another. Because earthquakes always occur along faults, the seismic hazard will be greater for population centres close to active fault zones, specifically in the southwest British Columbia. Figure B.2 illustrates three earthquake sources due to the Juan de Fuca fault in southwest British Columbia. These sources are:

1. relatively close to the surface in the North American Plate (continental crust);
2. deeper in the subducting Juan de Fuca Plate (oceanic crust);
3. along the boundary between the North American Plate and the subducting Juan de Fuca Plate (locked zone).

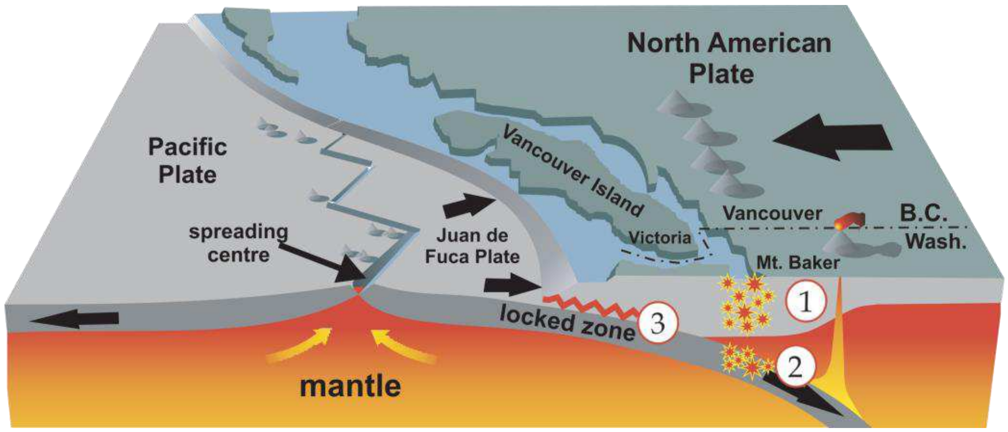

Figure B.2: Earthquakes in southwest British Columbia (NRCan 2009)

National Resources Canada (NRCan 2009) identifies the following *seismic zones* in Western Canada:

- *Offshore BC Region:* From northern Vancouver Island to the Queen Charlotte Islands (Haida Gwaii region), the oceanic Pacific Plate is sliding to the northwest at about 60 mm/year relative to North America.
- *Cascadia Subduction Zone:* West of Vancouver Island, and extending from the north tip of the Island to Northern California, the oceanic Juan de Fuca Plate is moving toward North America at about 20-50 mm/year.
- *St. Elias Region and the Southwestern Yukon:* The St. Elias region in southwest Yukon Territory, northwest British Columbia and southeast Alaska is one of the most seismically active areas in Canada. Here, the plate boundary between the giant Pacific and North American Plates is changing from one of transform (sliding past one another) to subduction (where the Pacific Plate is being forced beneath the Aleutian Islands to the northwest). This results in very rapid uplift rates (mountain *building*) of up to 30 mm/year.
- *Northern Cordillera:* The northern Rocky Mountain region is one of the most seismically active areas of Canada. It extends from northern Washington through Western Canada and central Alaska.
- *Southern Cordillera:* South of 60 degrees north latitude, between British Columbia and Alberta, *seismicity* in the interior and Rocky Mountain areas drops off rapidly.

- *Interior platform:* Seismic activity in the prairie region south of 60 degrees north latitude is predominantly confined to southern Saskatchewan in a zone that continues into Montana in the U.S.

B.1.2 **Eastern Canada**

In the eastern part of Canada, the cause of earthquakes is less well understood. There is no plate boundary and the zone lies entirely within the North American Plate, it is far from its active eastern and western boundaries in the mid-Atlantic, and just off British Columbia, respectively (NRCan 2009). Furthermore, very few locations of faults are known. Therefore, it is difficult to state where earthquakes are most likely to occur. The slow movement of the North American Plate away from the Mid-Atlantic Ridge may activate old zones of weakness and faults such as the St. Lawrence Valley (NRCan 2009).

Several significant historical earthquakes have occurred, such as a major earthquake in the Charlevoix, Quebec region near the mouth of the Saguenay River, in 1663. The 1663 earthquake is the first known damaging event to have occurred in Canada. It had an estimated magnitude of close to 7 and caused major landslides along the St. Lawrence River and several of its tributaries (NRCan 2009). Since 1663, the regions near the St. Lawrence and Ottawa Rivers have experienced five earthquakes with a magnitude of 6 or greater. Recent damaging seismic events occurred in La Malbaie in 1925 (magnitude 6.2 ± 0.3), Cornwall-Massena in 1944 (magnitude 5.6), and Saguenay in 1988 (magnitude 5.9). However, most earthquakes in the eastern part of the continent are events of smaller magnitude. Because of regional geologic differences, eastern and central North American earthquakes are felt at much greater distances than those in the western part, sometimes up to 1000 km away.

National Resources Canada (NRCan 2009) identifies the following seismic zones in Eastern Canada:

- Northeastern Ontario: This is a secondary region in Northern Ontario that lies north and east of Lakes Superior and Huron. It consists of the districts of Algoma, Sudbury, Cochrane, Timiskaming, Nipissing and Manitoulin. Northern Ontario has a very low level of seismic activity.

- *Southern Great Lakes:* This region has a low to moderate level of *seismicity* when compared to the more active seismic zones to the east, along the Ottawa River and in Quebec.

- *Western Quebec:* This Zone constitutes a vast territory that encloses the Ottawa Valley from Montreal to Temiscaming, as well as the Laurentians and Eastern Ontario. The urban areas of Montreal, Ottawa-Gatineau and Cornwall are located in this zone. Western Quebec is an active zone where an earthquake occurs every five days on average.

- *Charlevoix-Kamouraska:* Located some 100 km downstream from Quebec City in the estuary of the St. Lawrence River, the Charlevoix Seismic Zone (CSZ) is the most seismically active region in Eastern Canada. As most earthquakes occur under the St. Lawrence River, between Charlevoix County on the north shore and Kamouraska County on the south shore, this region is also often referred to as the Charlevoix-Kamouraska Seismic Zone.

- *Lower St. Lawrence:* Located some 400 km downstream from Quebec City in the estuary of the St. Lawrence River, the Lower St. Lawrence Seismic Zone (LSZ) is a seismically active region in Eastern Canada. As most earthquakes occur under the St. Lawrence River, between the regions of the Quebec North Shore and Lower St. Lawrence, this zone is sometimes referred to as the "Lower-St. Lawrence-Quebec North Shore" Seismic Zone.

- *Northern Appalachians:* The Northern Appalachians Seismic Zone includes most of New Brunswick and extends into New England down to Boston, in the U.S. The zone experiences a continuing low level of seismic activity, including many larger historic earthquakes in New Brunswick.

- *Laurentian Slope*: The Laurentian Slope Seismic Zone comprises an area off Canada's southeast coast, which includes the Grand Banks of Newfoundland. Earthquakes offshore have the potential of causing tsunamis.

B.1.3 **Arctic Canada**

In the Arctic, earthquakes also seem to be associated with older geological features. They may, however, also be related to stresses produced during uplift of the land following removal of the vast ice sheets from the last continental glaciation of this area (NRCan 2009). As compared to Western and Eastern Canada, the recorded history and instrumental data of seismic activity in the Arctic is very recent.

B.2 Seismic resistance of existing buildings

Buildings experience horizontal distortions when subjected to earthquake motions. When these distortions get large, the *damage* can be catastrophic. Therefore, most *buildings* are designed with seismic force-resisting systems (SFRSs) to resist the effects of seismic forces.

In many cases, SFRSs make a *building* stiffer and thus minimize the amount of lateral movement and consequently the amount of *damage*. SFRSs are usually capable of resisting only forces that result from ground motions parallel to them. However, the combined action of an SFRS along the width and length of a *building* can typically resist earthquake motion from any direction. SFRSs differ from *building* to *building*, because the type of system is controlled to some extent by the *building's* basic layout and structural elements. Basically, SFRSes consist of elements to resist axial (tension or compression), shear and bending stresses.

In wood-frame stud-wall *buildings*, plywood siding is typically used to prevent excessive lateral deflection. Without the extra strength provided by the plywood, walls would distort excessively or "rack," resulting in broken windows and stuck doors. In older wood frame *buildings*, this resistance to lateral loads is provided by either wood or steel bracing.

The SFRSes in modern steel *buildings* take many forms. Many types of diagonal bracing configurations have been used. Examples of the use of single diagonal bracing, double diagonal bracing, eccentric bracing and K-trusses are shown in Figure B.3. Moment-resisting steel frames are also capable of resisting lateral loads. In this type of construction, the connections between the beams and columns are designed to resist the rotation of the column relative to the beam. Thus, the beam and column work together and resist lateral movement by bending. This is different from the braced frame, where loads are resisted through tension and compression forces in the braces. Steel *buildings* are sometimes constructed with moment-resistant frames in one direction and braced frames in the other.

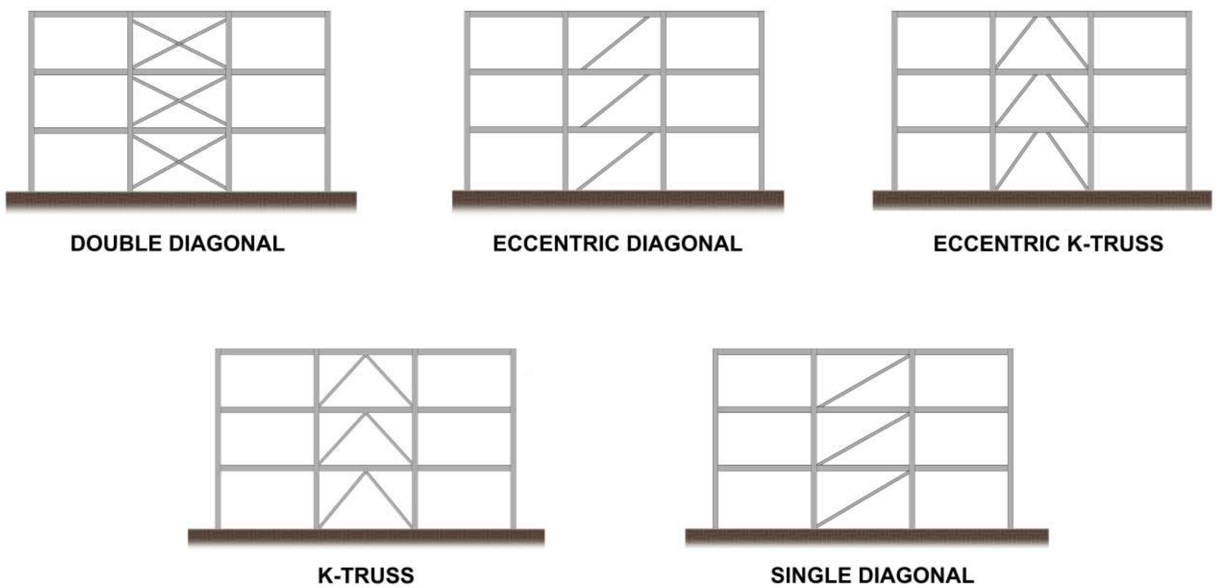

Figure B.3: Types of bracing

In concrete structures, shear walls are sometimes used to provide lateral resistance, in addition to moment-resisting frames. Exterior or interior shear walls are typically continuous reinforced concrete walls extending from the foundation to the roof of the *building*. They are interconnected with the rest of the concrete frame, and thus resist the relative displacement of floors. Shear walls can also be constructed of reinforced brick or reinforced hollow concrete block.

B.3 Seismic hazards in existing buildings

Structural and non-structural *damage* of *buildings* have the potential to cause significant harm to *occupants* and passersby, posing a major threat to *life safety*. It is, therefore, important to understand the nature of structural and non-structural *damage* and the associated seismic hazards in *existing buildings*.

Structural *damage* means degradation of the *building*'s structural support systems (i.e., vertical and lateral force-resisting systems), such as *building* frames and walls. Non-structural *damage* refers to any *damage* that does not affect the integrity of the structural support system. Examples of non-structural *damage* can be associated with failure of parapets, ornamentations, masonry partitions, heavy equipment such as elevators, lifelines in critical facilities, etc. The type of *damage* to be expected is a complex issue that depends, among other factors, on the structural type and age of the *building*, earthquake ground motion, soil conditions, the proximity of the *building* to adjacent *buildings*, condition of the *building*, and type of non-structural components. Hazards resulting from structural and non-structural *damage* will be discussed below.

B.3.1.1 Hazards due to structural damage

Damage of structural elements forming part of the SFRS reduces the seismic capacity of *buildings*, increasing the probability of partial or complete collapse. Unreinforced, inadequately reinforced or poorly constructed structural elements, such as poorly detailed and constructed unreinforced masonry walls, may represent a *life-safety* problem. Structural *damage* and differential movement due to seismic forces may lead to failure of electrical and gas lines or rupture of non-structural components containing *hazardous materials,* which could result in fire or explosion. Furthermore, structural *damage* may break water and communication lines that could potentially prevent post-earthquake emergency response.

Configuration, or the general vertical or horizontal shape of *buildings*, is an important factor in earthquake performance and *damage*. *Buildings* that have simple, regular, symmetric configurations generally display the best performance in earthquakes. Non-symmetric *buildings* tend to twist and shake laterally. Furthermore, *building* wings tend to act independently, which causes differential movements, cracking and other *damage*. Rotational motion introduces additional *damage*, especially at re-entrant or internal *building* corners. Rotation and associated *damage* can also exist in *buildings* with bracing systems, shear walls or moment-resisting frames that introduce an asymmetric distribution of lateral forces.

The expected *damage* in different *building* types is summarized as follows:

Wood light frame (WLF) and Wood post-and-beam (WPB)

- The most common type of structural *damage* in older WLF *buildings* results from a lack of connection between the superstructure and foundation.
- *Buildings* can slide off their foundations if they are not properly bolted to the foundation, resulting in major *damage* to the *building* as well as to plumbing and electrical connections.

- Many of the older wood-stud frame *buildings* either do not have foundations or have weak foundations constructed of unreinforced masonry or poorly reinforced concrete. These foundations have poor shear resistance and can fail.

- Post-and-beam *buildings* tend to perform well in earthquakes, if adequately braced. However, walls often do not have sufficient bracing to resist horizontal motion and thus they may deform excessively.

Steel moment frame (SMF) and steel light frame (SLF)

- The beam-column connections may not have the capacity to resist moments due to lateral earthquake loads. This may result in large plastic deformations and plastic hinges, increasing the potential of structural instability and formation of collapse mechanisms.

- Large plastic deformations may trigger non-structural *damage* to drift-sensitive elements such as interior partitions, equipment, and exterior cladding.

- The flexibility of steel frames may also result in *pounding* with adjacent *buildings* if separation gaps are inadequate.

Steel braced frame (SBF)

- Steel braced frames are expected to sustain smaller interstorey deformations or *damage* compared to unbraced steel frames.

- Braces with insufficient tensile capacity may rupture and braces with insufficient compressive capacity may buckle when subjected to high seismic demands. Brace failure may result in structure instability.

- Short sections of beams in eccentric braced frames are expected to deform significantly under major seismic forces and thereby dissipate a considerable portion of the energy. However, non-ductile or poorly detailed beams may fail as a result of moment and shear forces exceeding the design capacity.

Steel frame with concrete shear walls (SCW)

- Shear cracking and distress can occur around openings in concrete shear walls during earthquakes.

- Wall construction joints will potentially form weak planes, resulting in wall shear failure below expected capacity.

Steel frame with infill masonry shear walls (SIW)

- Because of the masonry infill, the structure tends to be very stiff. During major earthquakes, the stiffness of infill walls may be reduced due to substantial cracking and *deterioration*, transferring additional demands on the steel frames. Some of the walls may fail while others remain intact, which may result in severe vertical or horizontal *irregularities*. The hazard from falling masonry is significant, specifically with high-rise *buildings*.

- Infill walls tend to buckle and fall out-of-plane when subjected to strong lateral forces. These walls tend to be thin (around 230 mm maximum) and lack the shear strength to resist seismic forces.

- Soft or weak *storey*, where infill walls exist in the upper *storeys* but not on the ground floor. The difference in stiffness or strength creates a large demand on ground floor columns, causing structural *damage*.

- Non-structural components that are poorly anchored to structural members may be *damaged*.

- When 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 high-rise *buildings*) and can lead to *damage*.

Concrete moment frame (CMF)

- Insufficient reinforcement and lack of seismic detailing in non-ductile concrete moment frames can result in significant lateral deformations and brittle failure.

- Large tie spacing in columns can lead to a lack of concrete confinement or shear failure.

- Placement of inadequate reinforcing bar splices at the same location can lead to column failure.

- Insufficient shear strength in columns can lead to shear failure prior to the development of moment hinge capacity.

- Insufficient shear tie anchorage can lead to premature brittle failure in shear or compression.

- Lack of continuous beam reinforcement can result in hinge formation during load reversal.

- Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to failures.

- Relatively low stiffness of frames can lead to substantial non-structural *damage*.
- *Pounding damage* with adjacent *buildings* can occur.

Concrete shear wall (CSW)

- Some types of *damage* commonly observed in high-rise *buildings* are caused by vertical discontinuities, *pounding*, or irregular configuration.
- Shear cracking and distress can occur around openings in concrete shear walls during large seismic events.
- Shear failure can occur at wall construction joints usually at a load level below the expected capacity.
- Bending failure can result from insufficient chord steel lap lengths.

Concrete frame with infill masonry shear walls (CIW):

- Where unreinforced-masonry (URM) infill is present, a falling hazard exists. The failure mechanism of URM infill in a concrete frame is the same as steel frame infill.
- Infill walls tend to buckle and fall out-of-plane when subjected to strong lateral out-of-plane forces.
- Veneer masonry and partitions are usually poorly anchored and detach easily from structural members.

Precast concrete wall (PCF)

- Many precast concrete wall *buildings* do not have sufficiently strong connections or anchors between walls, roofs and floor diaphragms. During an earthquake, weak anchors pull out of walls, causing floors or roofs to collapse. The connections between concrete panels are also vulnerable to failure. Without these connections, the *building* loses much of its lateral force-resisting capacity.
- Since 1975, precast construction practices have changed in seismic areas of Canada, requiring positive wall-diaphragm connections. However, a large number of older, pre-1970s precast concrete wall *buildings* still exist, and have not been retrofitted to correct this wall-anchor defect.
- Precast concrete wall *buildings* are a primary source of seismic hazards in areas of low or moderate *seismicity* where inadequate wall anchor details are employed. Severe ground-shaking in such areas may produce major *damage*.

Precast concrete frame (PCF)

- The earthquake performance of this structural type varies greatly and is sometimes poor. This type of *building* can fail if the connections between the prefabricated structural elements have insufficient strength and ductility.
- Accumulated stresses or gaps can result because of shrinkage and creep.
- Loss of vertical support can occur due to inadequate bearing area or insufficient connection between floor diaphragm elements, beams, wall panels, and columns.
- Corrosion of metal connectors between prefabricated elements can occur.
- Connections at bases of precast columns and wall panels may be inadequate.

Reinforced-masonry bearing walls with wood or metal deck diaphragms (RML):

- A major problem is workmanship quality control during construction. Poor construction practices may result in ungrouted and unreinforced walls.
- Where construction practice is inadequate, insufficient reinforcement can be responsible for extensive wall *damage*. The lack of positive connection between the floor or roof diaphragm and wall is also a problem.

Reinforced-masonry bearing walls with concrete diaphragms (RMC)

- Poorly anchored and connected precast concrete diaphragms can be responsible for earthquake-related *damage*.

Unreinforced-masonry bearing-wall (URM)

- 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.
- Walls, parapets, and cornices not adequately anchored to floors tend to fallout. The collapse of bearing walls can 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 may exhibit questionable performance.
- Some of these *buildings* have *storey* heights and thin walls. This condition, especially in non-load-bearing walls, can potentially 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.

- The mortar used in old *buildings* is often made of lime and sand, with little or no cement, and has very little shear strength. Bearing walls will be heavily *damage*d and may collapse under large loads.

- Because most of the floor diaphragms in these *buildings* 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 low to moderate *seismic zones*.

Manufactured homes (MH)

- Foundation systems such as metal tripods or concrete blocks designed only for carrying gravity loads do not provide stable support to manufactured homes in case of moderate- to very-high-intensity earthquakes. These supports, which are not typically secured to the ground, may move and cause the manufactured home to drop, resulting in structural and non-structural component *damage* that may significantly cause harm to *occupants*.

B.3.1.2 Hazards due to non-structural damage

Investigation of non-structural hazards can be time-consuming and should be done during detailed seismic evaluation of the particular *building*. However, the screening process should include a review of critical hazardous non-structural components.

Exterior non-structural components

a) Exterior cladding, veneers and window glass can fall onto the storefronts, streets, sidewalks and adjacent properties if their connectors to the *building* structure have insufficient strength or sometimes ductility. In older *buildings*, the veneers are either insufficiently attached or have poor quality mortar, which often results in veneer peeling off during moderate or large earthquakes.

b) Parapets, cornices, ornamentations and other appendages are very vulnerable during earthquakes. Falling masonry parapets have caused death to passing pedestrians during past earthquakes. These architectural features are usually non-structural and may be subject to substantial motion amplifications that can trigger failure.

c) Unreinforced masonry chimneys may also represent a life-safety problem. They are often inadequately attached to the *building* and therefore fall when strongly shaken. Chimneys of reinforced masonry generally perform well.

Interior non-structural components

a) Partitions made of different types of material (masonry and tile, studs and gypsum board or plaster, demountable partitions of metal, wood, or glass) may be destroyed, overturned, cracked, or separated from the remainder of partitions. However, only unsupported heavy partitions are considered life-threatening.

   It is also important to consider partitions adjacent to *building* entrances and exits, since failure of these partitions may prevent escape from the *building*, which is a threat to *life safety*.

b) Mechanical and electrical equipment items, such as pumps, fans, piping, ducts and electrical panels, when well attached to the walls or floors, generally perform well during earthquakes. However, rigid piping and their supports or hinges may fail. Electrical or mechanical equipment may overturn. Well-attached equipment can move horizontally and impact other items, often causing them to cease function. The danger of a fire starting due to such displacements should not be overlooked.

   Elevator counterweights may become separated from rails, causing *damage* to structural elements, cables and cabs, and leaving the elevators inoperative.

c) Interior water storage tanks and pressure vessels may fall from their supports and cause severe *damage* to floors and other structural elements. They are, however, only considered life-threatening when they carry *hazardous* (hot or corrosive) *materials* or when their failure causes other main structural elements to fail.

B.4 Seismic vulnerability of existing buildings

Although *buildings'* earthquake behaviour cannot be precisely predicted, there are general trends that have been observed in past earthquakes. For example, steel *buildings* perform significantly better than those built of unreinforced masonry. New *buildings* generally sustain less *damage* than older *buildings* designed to earlier seismic codes. Simple, regular, and symmetric *buildings* display the best performance in earthquakes. From a *life-safety* perspective, seismic vulnerabilities of *existing buildings* need to be identified and strengthened or removed.

Major factors that contribute to the seismic vulnerability of *existing buildings* are summarized and discussed as follows:

1. Vertical *irregularity*: Vertical *irregularities* adversely affect the seismic performance of a *building* by concentrating demands at certain floor levels or elements. Concentrated demands can lead to *damage*, failure and, in some cases, collapse.

2. Horizontal *irregularity*: Horizontal *irregularities* can significantly reduce a gravity load-carrying system's capacity, leading to partial or total collapse.

3. *Site Class*: The soil profile, soil properties, and geologic structure of the ground at the *building* site are important factors that influence seismic vulnerability. Soft soils can significantly amplify long-period spectral response accelerations.

4. *Geologic hazards*: *Geologic hazard*s such as *liquefaction*, *landslide potential*, and *surface fault rupture* have the potential to significantly increase a *building*'s risk of sustaining *damage* or collapsing during an earthquake.

5. *Pounding*: Adjacent *buildings* can pose *pounding* threat to the *building* of interest if the separation distance between them is less than a specified minimum separation distance defined as a function of *seismicity*. The *pounding* problem is prevalent in downtown areas where *buildings* were constructed before sufficient separation distances were established. The other problem from adjacent *buildings* is *falling hazards*. Unanchored exterior walls, unbraced parapets, chimneys or smokestacks, tanks, signs or any other *building* components from taller adjacent *buildings,* which, if dislodged, could fall onto the *building* being screened or block its major egress routes.

6. Pre-code design: *pre-code buildings* are expected to exhibit significantly worse performance than *buildings* that have been designed and constructed according to seismic code requirements, because they have no or minimum lateral resistance capacity to resist earthquake loadings.

7. *Building deterioration* and *damage*: *Deterioration* or *damage* of structural elements and non-structural components that could pose hazards to *life safety* can have a significant impact on the expected performance of an *existing building* and therefore needs to be captured in the screening process.

8. Non-structural components posing hazards to *life safety*: Non-structural components such as parapets, chimneys and heavy claddings can pose hazards to *life safety* if not adequately anchored to the *building*. In addition, the failure or overturning of non-structural components that contain *hazardous material* can pose a *life safety* threat.

B.5 Characteristics and earthquake performance of sixteen model building types

This section provides additional information about each of the sixteen *model building types* used in the *Level 2 – SQST*, including detailed descriptions of their characteristics, typical seismic performance, and earthquake *damage*.

B.5.1 **Wood frames**

Wood light frame (WLF)

Engineered Woodd Light Frame (WLF) *buildings* are typically apartment, commercial or office buildings of up to 6 storeys in *building* height, or having a *building* area exceeding 600 m^2 (see

Figure B.4.) WLF *buildings* are covered by Part 4 of the NBC and designed in accordance with the CSA O86 Engineering Design in Wood.

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.

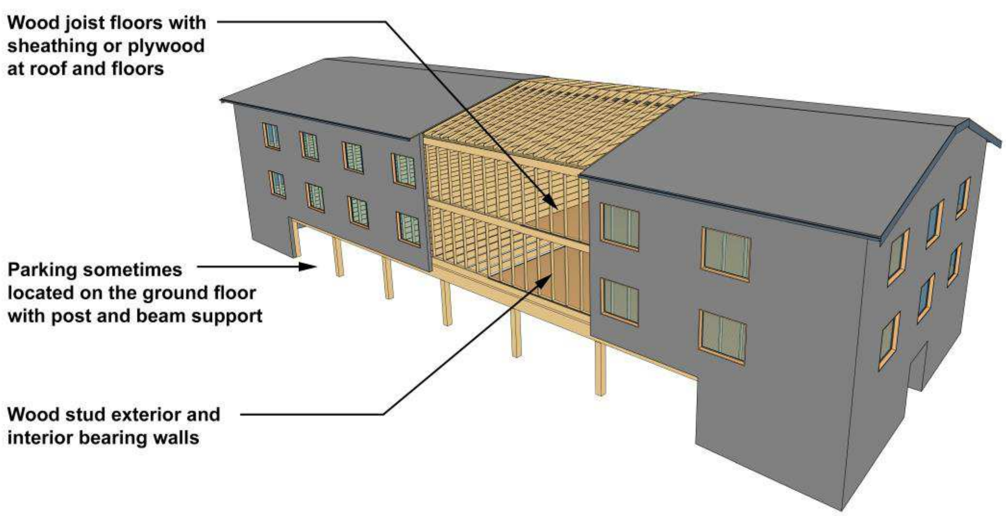

Figure B.4: Schematic illustration of an engineered WLF *building* (Figure adapted from FEMA 547, FEMA, 2006)

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 (see Figure B.5). 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.5: Schematic illustration of an engineered WPB *building* (Figure adapted 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.

Typical earthquake damage

Stud-wall *buildings* have performed very well in past earthquakes due to the low height, light weight, and redundancy in the system. Cracks in the plaster and stucco may occur, but such damage is typically considered as non-structural. The most common type of structural damage in older *buildings* results from a lack of connection between the superstructure and the foundation. Older (usually pre-1940) houses were often not adequately connected to their foundation which results in the building sliding off the foundation during an earthquake. Houses and most *buildings* constructed in Canada since the early 1970s have been required to meet the seismic safety requirements in the NBC.

Likewise, unreinforced masonry chimneys represent a life-safety problem. They are often inadequately tied to the building and therefore fall when strongly shaken, causing damage to other parts of the building. On the other hand, chimneys of reinforced masonry generally perform well.

Post-and-beam *buildings* tend to perform well in earthquakes, if adequately braced. However, frames often do not have sufficient bracing to resist horizontal motion and thus they may deform excessively.

Post-and-beam *buildings* often have different types of external walls, including masonry or fake stone veneers, which may represent a type of falling hazards.

B.5.2 **Steel frames**

Steel moment frame (SMF)

This *building* type has a frame of steel columns and beams (see Figure B.6). 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.

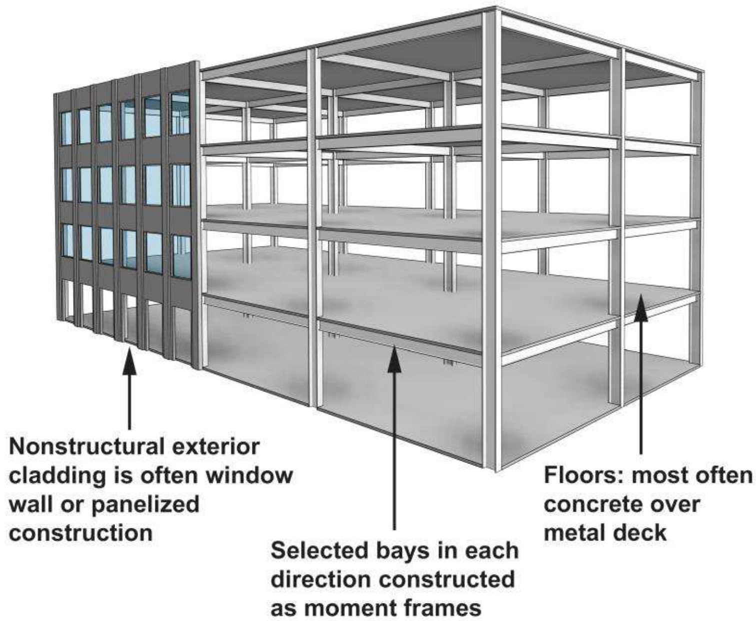

Figure B.6: Schematic illustration of a SMF *building* (Figure adapted from FEMA 547, FEMA, 2006)

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 terra cotta tiles. (Refer to steel frame with infill masonry shear walls – SIW for a detailed discussion.) Other frame buildings for this period are encased in concrete. Both wood and concrete floor or roof diaphragms are common for these older buildings.

Steel braced frame (SBF)

Steel braced frame structures (Figure B.7) 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.7: An example of an SBF *building*

In concentrically braced frames, the lateral forces or loads are resisted by the tensile and compressive strength of the bracing (see Figure B.8). In eccentrically braced frames, the bracing is slightly offset from the main beam-to-column connections, and the short section of beam is expected to deform significantly by bending under major seismic forces and thereby dissipate a considerable portion of the vibrating building's energy.

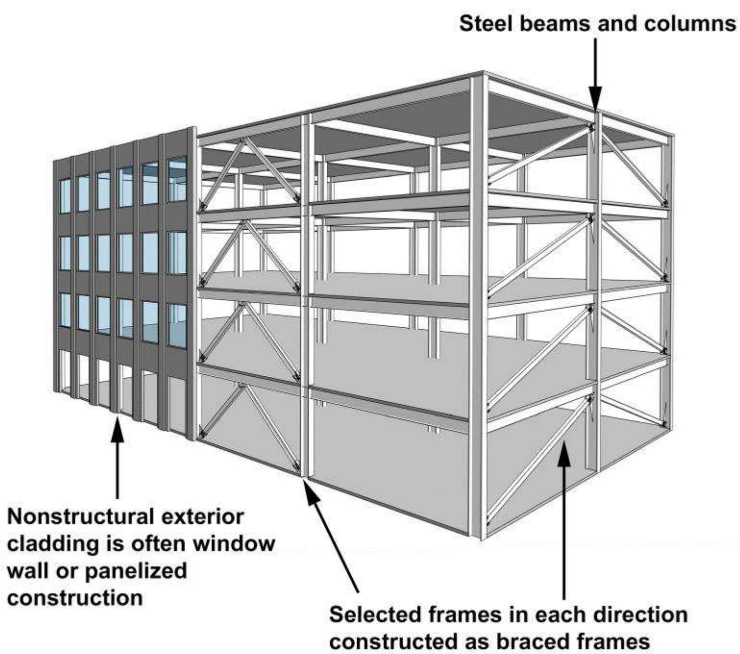

Figure B.8: Schematic illustration of a SBF *building* concentrically braced frames (Figure adapted from FEMA 547, FEMA, 2006)

Typical earthquake damage

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 U.S., these buildings have performed quite well, and probably will not collapse unless subjected to extremely severe ground shaking. The 1994 Northridge and 1995 Kobe earthquakes showed that steel frame buildings (in particular SMF moment-frame) were vulnerable to severe earthquake damage. Although none of the damaged buildings collapsed, they were rendered unsafe until repaired. Damage was in the form of broken welded connections between the beams and columns. Cracks in the welds began inside the welds where the beam flanges were welded to the column flanges. These cracks, in some cases, broke the welds or propagated into the column flange, "tearing" the flange. Damage was found in buildings that experienced ground accelerations of approximately 0.2 g or greater. Possible damage includes:

- Non-structural damage to elements such as interior partitions, equipment and exterior cladding, resulting from excessive deflection in frame structures;
- Falling of cladding and exterior finish material if inadequately or incorrectly connected;
- Permanent displacements caused by plastic deformation of structural members;
- Damage due to pounding with adjacent structures.

B.5.3 **Steel light frame (SLF)**

Most steel light frame buildings existing today were built after 1950 (see Figure B.9). 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 metal or asbestos-reinforced concrete siding, often corrugated.

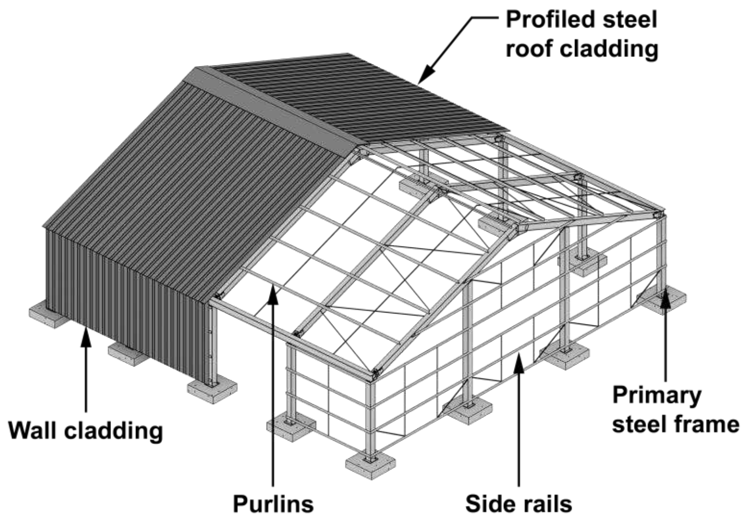

Figure B.9: Schematic illustration of an SLF *building* (Figure adapted from FEMA 547, FEMA, 2006)

To identify this construction type, the screener should look for the following characteristics:

- Light metal buildings are typically characterized by industrial corrugated sheet metal or asbestos-reinforced cement siding. The term, "metal building panels" should not be confused with "corrugated sheet metal siding." The former are prefabricated cladding units usually used for large office buildings. Corrugated sheet metal siding is a thin sheet material usually fastened to purlins, which in turn span between columns. If this sheet cladding is present, the screener should carefully examine the fasteners used. If the heads of sheet metal screws can be seen in horizontal rows, the building is most likely a light metal structure.

  Because the typical structural system consists of moment frames in the transverse direction and frames braced with diagonal steel rods in the longitudinal direction, light metal buildings often have low-pitched roofs without parapets or overhangs. Most of these buildings are prefabricated, so the buildings tend to be rectangular in plan, with few corners.

- These buildings generally have only a few windows, as it is difficult to detail a window in the sheet metal system.

- The screener should look for signs of a metal building, and should knock on the siding to see if it sounds hollow. Door openings should be inspected for exposed steel members. If a gap or light can be seen where the siding meets the ground, it is certainly a light metal or wood frame. For the best indication, an interior inspection will confirm the structural frame, because most of these buildings do not have interior finishes.

Typical earthquake damage

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:

- Insufficient capacity of tension braces can lead to their elongation and, in turn, to building damage;
- Inadequate connection to the foundation can allow the building columns to slide;
- Loss of cladding.

B.5.4 Steel frame with concrete shear walls (SCW)

Construction of this structural type (see Figure B.10) is similar to that of a 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 lateral forces are resisted by concrete shear walls.

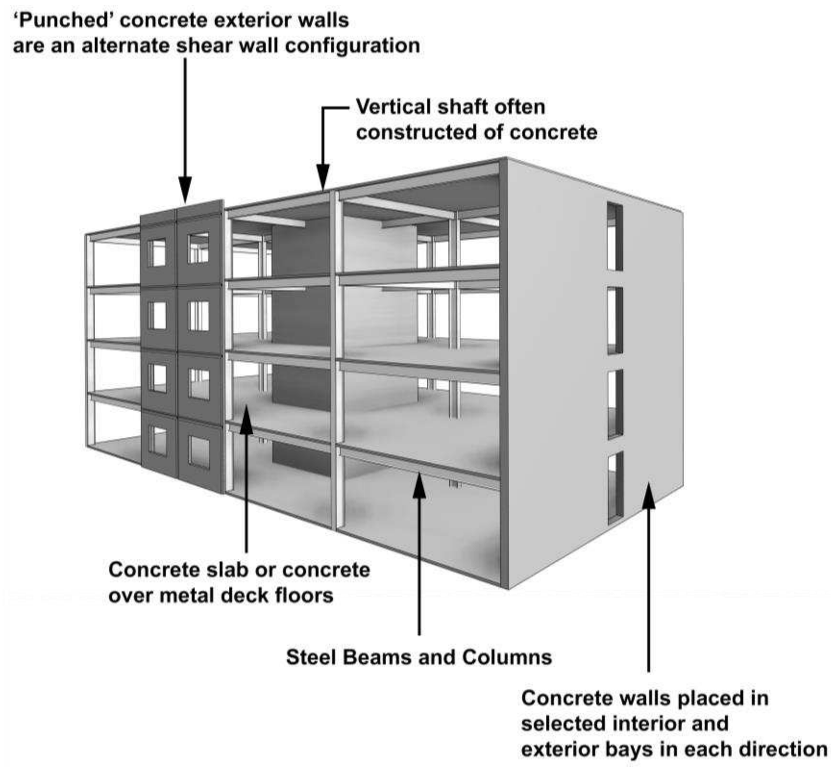

Figure B.10: Schematic illustration of a SCW *building* (Figure adapted from FEMA 547, FEMA, 2006)

It is usually difficult to distinguish between a steel frame with concrete shear walls and one without, just from a visual inspection of the building, because interior shear walls will often be covered by interior finishes and will look like interior non-structural partitions. It is suggested that the screener review the design and construction drawings available prior to the site visit.

Typical earthquake damage

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 steel buildings. Some typical types of damage:

- Shear cracking and distress around openings in concrete shear walls;
- Wall construction joints can be weak planes, resulting in wall shear failure below expected capacity;
- Insufficient lap lengths in vertical reinforcing steel leading to wall bending failures.

B.5.5 Steel frame with infill masonry shear walls (SIW)

This type of construction (see Figure B.11) 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. Because the steel frames in older buildings are covered by URM for fire protection, it is easy to confuse this type of building with URM bearing wall structures. Furthermore, because steel columns are relatively thin, they may be hidden in the walls.

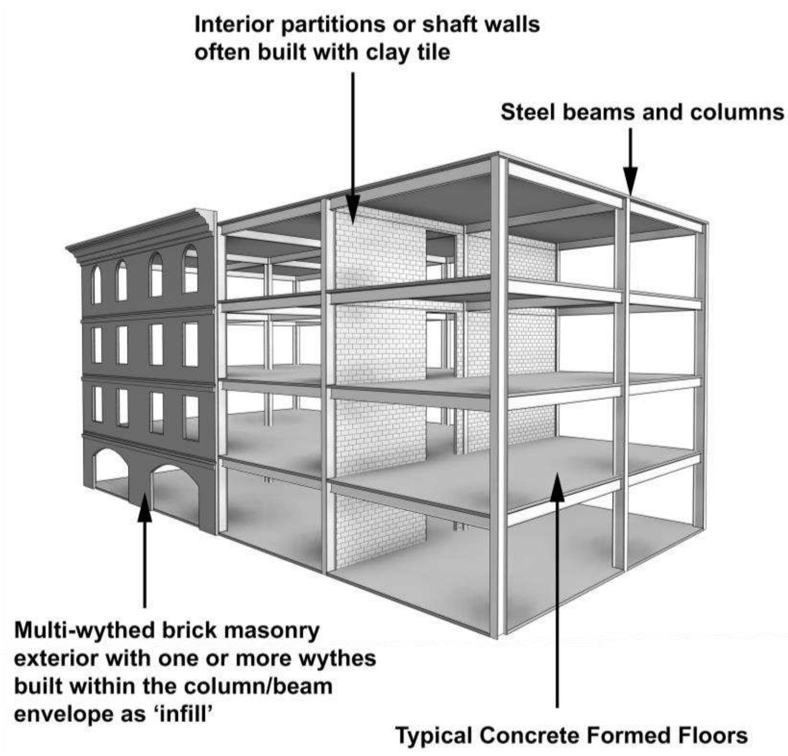

Figure B.11: Schematic illustration of an SIW *building* (Figure adapted from FEMA 547, FEMA, 2006)

An apparently solid masonry wall may enclose a series of steel columns and girders. These infill walls are usually two or three wythes (a wythe is a term denoting the width of one brick) thick. As a result, header bricks will sometimes be present and thus mislead the screener into thinking the building is a URM bearing wall structure, rather than an SIW structure. In these buildings, the infill and veneer masonry are usually exposed. However, masonry may be obscured by cladding in buildings, especially those that have undergone renovation.

When a masonry building is encountered, the screener should first attempt to determine if the masonry is reinforced by checking construction date; however, that is only a rough guide. A clearer indication of a steel frame structure with URM infill is when the building exhibits the characteristics of an SMF or SBF frame structure. One can assume all frame buildings clad in brick and constructed prior to about 1953 (pre-code year for all model building types except the precast concrete wall building type) are of this type.

Older frame buildings may be of several types: steel frame encased with URM, steel frame encased with concrete, and concrete frame. Sometimes older buildings have decorative cladding such as terra cotta or stone veneer. Veneers may obscure all evidence of URM. In that case, the structural type cannot be determined. However, if there is evidence that a large amount of concrete is used in the building (for example, a rear wall constructed of concrete), then it is unlikely that the building has URM infill.

When the screener cannot be sure if the building is a frame or has bearing walls, two clues may help: the thickness of the walls and the height. Because infill walls are constructed of two or three wythes of bricks, they should be approximately 230 mm thick (two wythes). Furthermore, the thickness of the wall will not increase in the lower stories, because the structural frame is carrying the load. For buildings over six stories tall, URM is infill or veneer, because URM bearing wall structures are seldom this tall and, if so, they will have extremely thick walls in the lower stories.

Typical earthquake damage

In major earthquakes, infill walls may suffer substantial cracking and deterioration from in-plane or out-of-plane deformation, thus reducing in-plane wall stiffness. This in turn puts additional demand 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 can be summarized as follows:

- Infill walls tend to buckle and fall out-of-plane when subjected to strong lateral forces. Because infill walls are non-load-bearing, they tend to be thin (around 230 mm maximum) and do not have additional shear strength because of compression from above.

- Veneer masonry around columns or beams is usually poorly anchored to the structural members and can disengage and fall.
- Interior infill partitions and other non-structural elements can be severely damaged and collapse.
- 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.
- Local failure of the steel frame itself when earthquake forces are very high. Connections between members are usually not designed for high lateral loads (except in tall buildings) and can lead to damage. Although complete collapse has seldom occurred, it cannot be ruled out.

B.5.6 **Concrete moment frame (CMF)**

Concrete moment frame buildings consist of concrete beams and columns that resist both lateral and vertical loads (see Figure B.12). There may be a few bays infilled with masonry, but if there is more extensive infill, it would be categorized as a concrete moment frame with infill masonry shear walls (CIW). A fundamental factor in the seismic performance of concrete moment frames is the presence or absence of ductile detailing. Two construction subtypes fall under this category: (a) non-ductile reinforced-concrete frames without reinforced infill walls, and (b) 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.

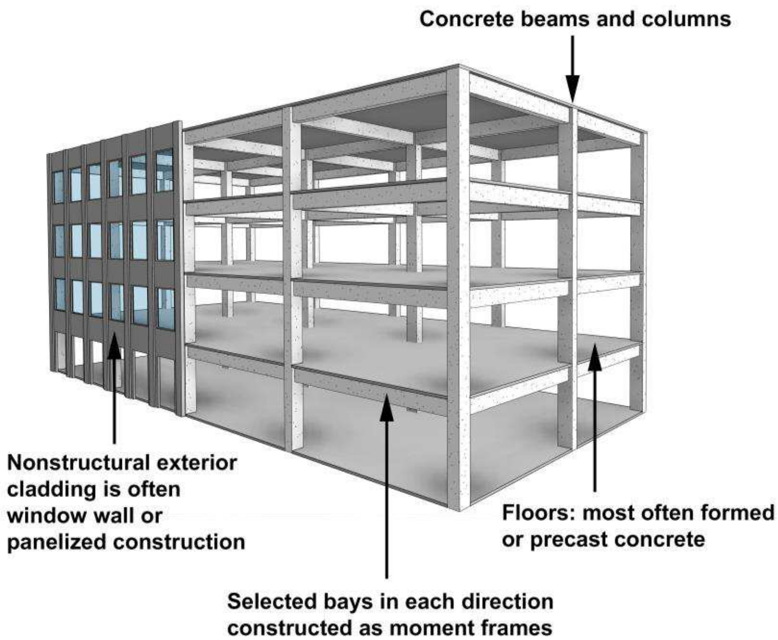

Figure B.12: Schematic illustration of a CMF *building* (Figure adapted from FEMA 547, FEMA, 2006)

Ductile detailing refers to the presence of special steel reinforcing within concrete beams and columns. Special reinforcement provides confinement of the concrete, permitting good member performance beyond elastic capacity, primarily in bending. Due to this confinement, crushing of concrete is delayed and the concrete retains its strength for more loading cycles (i.e., ductility is increased). Refer to Figure B.13 for an extreme example of ductility in concrete.

Figure B.13: Extreme example of ductility in concrete, 1994 Northridge earthquake (http://welldesignedfaith.net/2014/09/12/of-moment-frames-and-church-hymns/)

In many low-seismicity areas, non-ductile reinforced concrete frames are still allowed to be constructed. 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 are generally more massive than steel frame buildings, are under-reinforced (i.e., have insufficient reinforcing steel embedded in the concrete) and display low ductility.

It is usually difficult to distinguish between a CMF building type and SMF building type, unless the structural concrete has been left somewhat exposed. Although a steel frame may be encased in concrete and appear to be a concrete frame, this is seldom the case for modern buildings (post 1953). For the purpose of field screening, it can be assumed that all exposed concrete frames are concrete and not steel frames.

Typical earthquake damage

Under high amplitude cyclic loading, lack of confinement will result in rapid degradation of non-ductile concrete members, with ensuing brittle failure and possible building collapse.

Some typical problems include:

- Large tie spacing in columns can lead to a lack of concrete confinement or shear failure.
- Placement of inadequate reinforcing bar splices at the same location can lead to column failure.
- Insufficient shear strength in columns can lead to shear failure prior to the development of moment hinge capacity.
- Insufficient shear tie anchorage can lead to premature brittle failure in shear or compression.
- Lack of continuous beam reinforcement can result in hinge formation during load reversal.
- Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to failures.
- The frame's relatively low stiffness can lead to substantial non-structural damage.
- Pounding damage with adjacent buildings can 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.5.7 **Concrete shear walls (CSW)**

This category consists of buildings with a concrete wall structural system or frame structures with shear walls (see Figure B.14). 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.

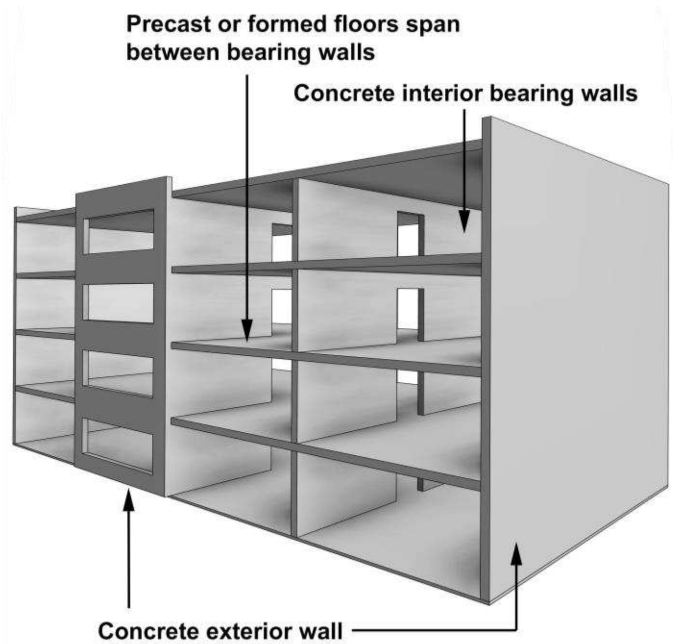

Figure B.14: Schematic illustration of a CSW *building* (Figure adapted from FEMA 547, FEMA, 2006)

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.

Frame structures with interior shear walls are difficult to positively identify. When the building is clearly a box-like bearing wall structure, it is probably a shear wall structure. Concrete shear wall buildings are usually cast in place.

The screener should look for signs of cast-in-place concrete. In concrete bearing wall structures, wall thickness ranges from 150 mm to 250 mm and is thin in comparison to that of masonry bearing wall structures.

Typical earthquake damage

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:

- Shear cracking and distress can occur around openings in concrete shear walls during large seismic events (see Figure B.15).
- Shear failure can occur at wall construction joints usually at a load level below the expected capacity.
- Bending failure can result from insufficient chord steel lap lengths.

Figure B.15: Shear wall damage (http://www.reidmiddleton.com/reidourblog/earthquake-reconnaissance-trip-chile/)

B.5.8 Concrete frame with infill masonry shear walls (CIW)

This category consists of buildings with a concrete frame with unreinforced-masonry or reinforced-masonry infill walls (see Figure B.16 and Figure B.17). These *buildings* tend to have larger members, although the amount and detailing of reinforcement is more in question. These concrete frames have been used for commercial and industrial structures, and are generally more than three storeys tall.

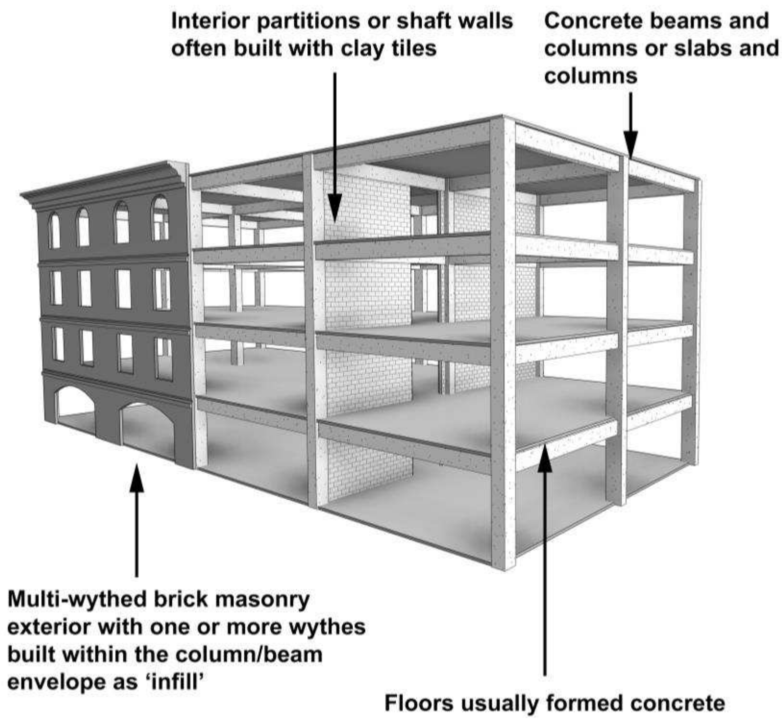

Figure B.16: Schematic illustration of a CIW *building* (Figure adapted from FEMA 547, FEMA, 2006)

Figure B.17: An example of a CIW *building* showing concrete frame and infill masonry shear walls (https://thetorontoblog.com/)

The first step in identifying building type is to determine whether the structure is old enough to contain URM. In contrast to steel frames with URM infill, concrete frames with URM infill usually show clear evidence of the concrete frames. This is particularly true for industrial buildings and can usually be observed at the side or rear of commercial buildings. The concrete columns and beams are relatively large and are usually not covered by masonry, but left exposed.

The URM may not be readily identified for commercial buildings with large windows on all sides; these buildings may have interior URM partitions. Another difficult case occurs when the exterior walls are covered by decorative tile or stone veneer. Infill material can be URM or a thin concrete infill.

Typical earthquake damage

The hazards of these older buildings are similar to and perhaps more severe than those of the newer frames. Where unreinforced masonry (URM) infill is present, a falling hazard exists. The failure mechanism of URM infill in a concrete frame is the same as steel frame infill.

B.5.9 Precast concrete walls (PCW)

Most PCW buildings use the tilt-up construction method. In traditional tilt-up buildings (see Figure B.18 and Figure B.19), concrete wall panels are cast on the ground and then tilted upward into their final position. 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. They are typically one and sometimes two storeys high, and typically have a simple rectangular plan. These buildings tend to be industrial or office buildings.

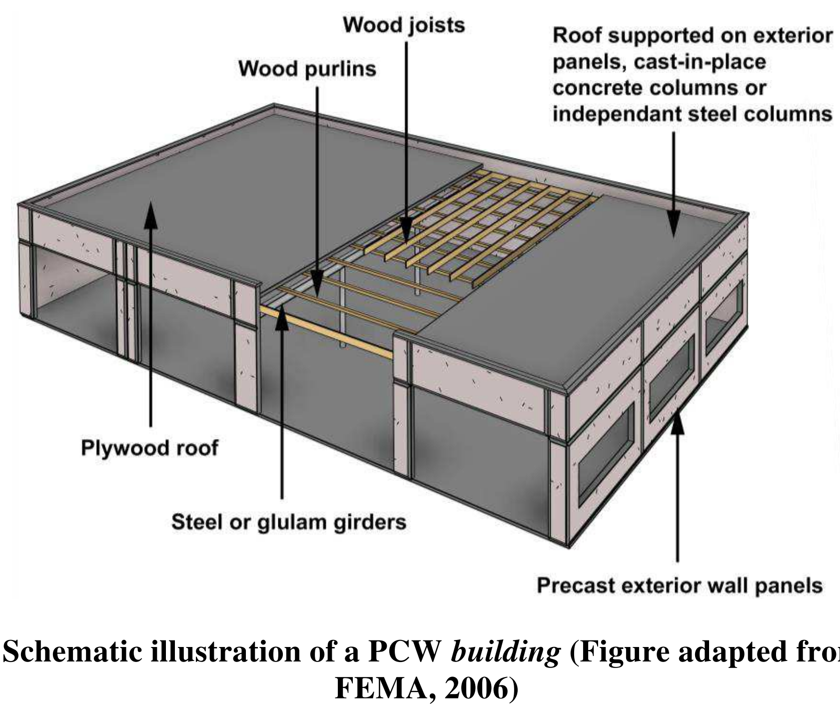

Figure B.18: Schematic illustration of a PCW *building* (Figure adapted from FEMA 547, FEMA, 2006)

Figure B.19: Example of a PCW building (http://www.tilt-up.org/projects/profile/?id=5484)

Many tilt-up buildings do not have sufficiently strong connections or anchors between the walls, roof and floor diaphragms. During an earthquake, weak anchors pull out of walls, causing floors or roofs to collapse. The connections between concrete panels are also vulnerable to failure. Without these connections, the building loses much of its lateral force-resisting capacity. For these reasons, many tilt-up buildings were damaged in the 1971 San Fernando earthquake.

Typical earthquake damage

The major defect in existing tilt-up buildings is a lack of positive anchorage between wall and diaphragm. During an earthquake, weak anchorage can break the ledgers, causing panels to fall and supported framing to collapse. When mechanical anchors are used, they pull out of the walls or split the wood members to which they were attached, causing the floors or roofs to collapse (see Figure B.20). Connections between the concrete panels are also vulnerable to failure. Without these connections, the building loses much of its lateral force-resisting capacity. For these reasons, many tilt-up buildings were damaged in the 1971 San Fernando earthquake.

Figure B.20: Collapse of a PCW building
(http://www.reidmiddleton.com/reidourblog/earthquake-reconnaissance-trip-chile/)

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 tilt-up buildings still exist and have not been retrofitted to correct this wall-anchor defect. Damage to these buildings was observed again 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.5.10 **Precast concrete frame (PCF)**

Precast concrete frame construction, first developed in the 1930s, was not widely used until the 1960s. The precast frame (see Figure B.21) 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 "T" sections, cross-shaped or arches, and are often more than one storey in height. Beams are often "T" section and double "T" sections, or rectangular sections. Pre-stressing of the members, including pre-tensioning and post-tensioning, is often used. Some typical characteristics are summarized in the following:

- Precast concrete, in general, is of a higher quality and precision compared to cast-in-place concrete. It is also available in a greater range of textures and finishes. Many newer concrete and steel buildings have precast concrete panels and column covers as an exterior finish (see Figure B.22). Thus, the presence of precast concrete does not necessarily mean that it is a precast concrete frame.

- Precast concrete frames are, in essence, post and beam construction in concrete. Therefore, when a concrete structure displays the features of a post-and-beam system, it is most likely a precast concrete frame. It is usually not economical for a conventional cast-in-place concrete frame to look like a post-and-beam precast system. Features of a precast concrete post-and-beam system include:
    - exposed ends of beams and girders that project beyond their supports or project away from the building surface;
    - the absence of small joists;
    - beams sitting on top of girders rather than meeting at a monolithic joint (see Figure B.23).

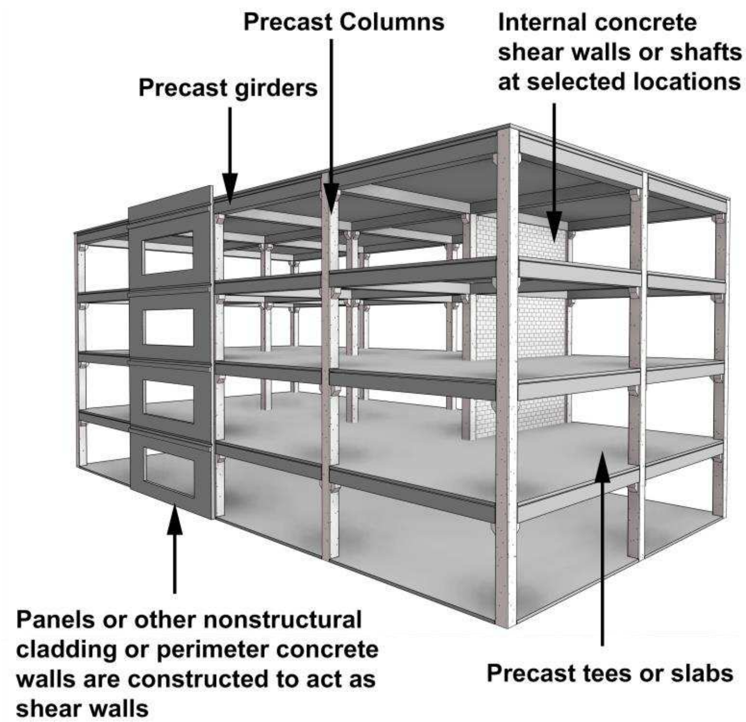

Figure B.21: Schematic illustration of a PCF *building* (Figure adapted from FEMA 547, FEMA, 2006)

Figure B.22: Typical precast concrete cover on a steel or concrete moment frame (https://www.buildingdesignindex.co.uk/entry/110238/BCM-GRC/GClad-glassfibrereinforced-concrete-column-casing/)

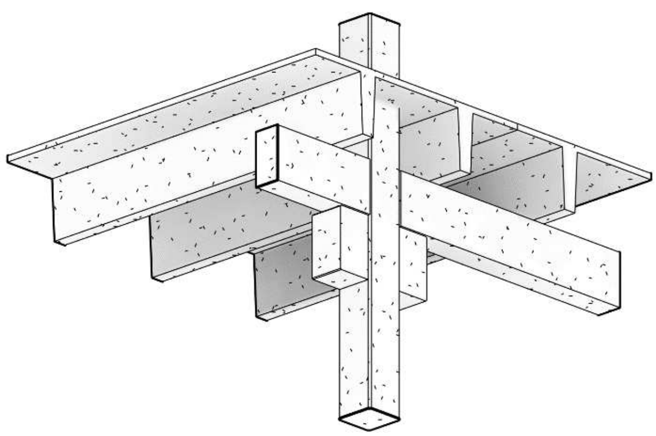

Figure B.23: Exposed double-T sections and overlapping beams are indicative of precast frames

The presence of precast structural components is usually a good indication of PCF building type, although these components are also used in mixed construction. Precast structural components come in a variety of shapes and sizes. The most common types are sometimes difficult to detect from the street. Less common, but more obvious examples include the following:

- **T-sections or double T's:** These are deep beams with thin webs and flanges and with large span capacities.
- **Cross or T-shaped units of partial columns and beams:** These are structural units for constructing moment-resisting frames. They are usually joined together by field-welding steel connectors cast into the concrete. Joints should be clearly visible at the mid-span of the beams or the mid-height of the columns. See Figure B.24.
- **Precast arches:** Precast arches and pedestals are popular in the architecture of these buildings.
- **Column:** When a column displays a precast finish without an indication that it has a cover (i.e., no vertical seam can be found), the column is likely to be a precast structural column.

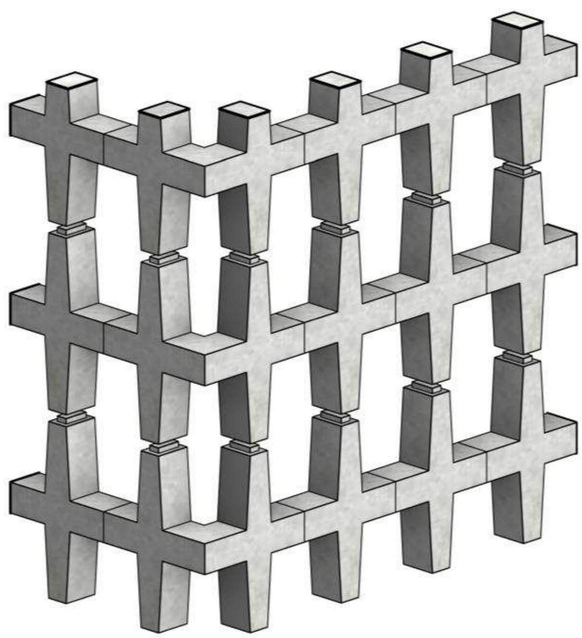

Figure B.24: Precast structural cross; installation of joints are at sections where bending is minimal during high seismic demand

It is possible that a precast concrete frame may not show any of the above features. Structures of this type that employ cast-in-place concrete shear walls for lateral load resistance should be treated as concrete shear walls (CSW).

Typical earthquake damage

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. Because structures of this type often employ cast-in-place concrete or reinforced masonry (brick or block) shear walls for lateral-load resistance, they experience the same types of damage as other shear wall building types. Some of the problem areas specific to precast frames:

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

B.5.11 **Reinforced masonry**

Reinforced masonry buildings are mostly low-rise structures with perimeter bearing walls, often with wood or light metal diaphragms (RML building type, Figure B.25), although precast concrete (RMC building type, Figure B.26) 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. Reinforced masonry structures are usually basically rectangular structures (see Figure B.27).

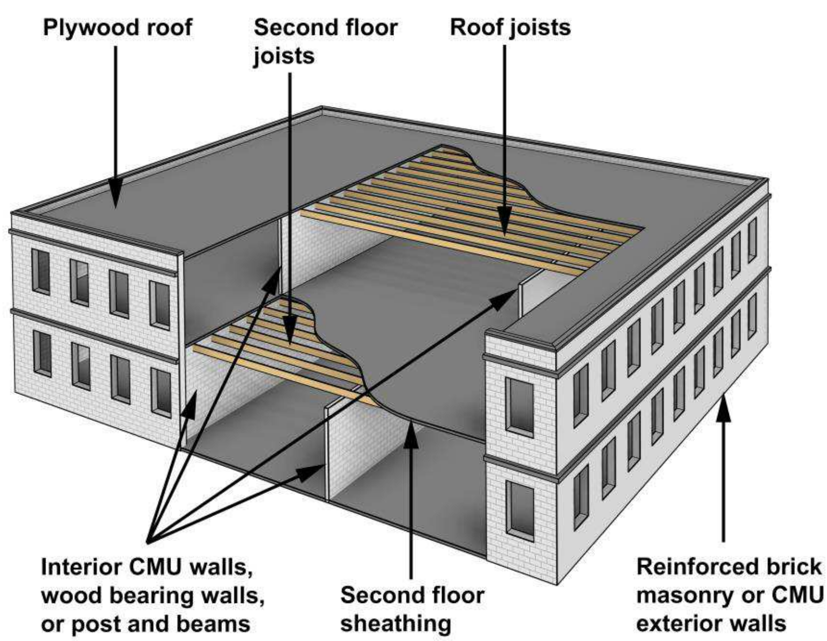

Figure B.25: Schematic illustration of a RML *building* (Figure adapted from FEMA 547, FEMA, 2006)

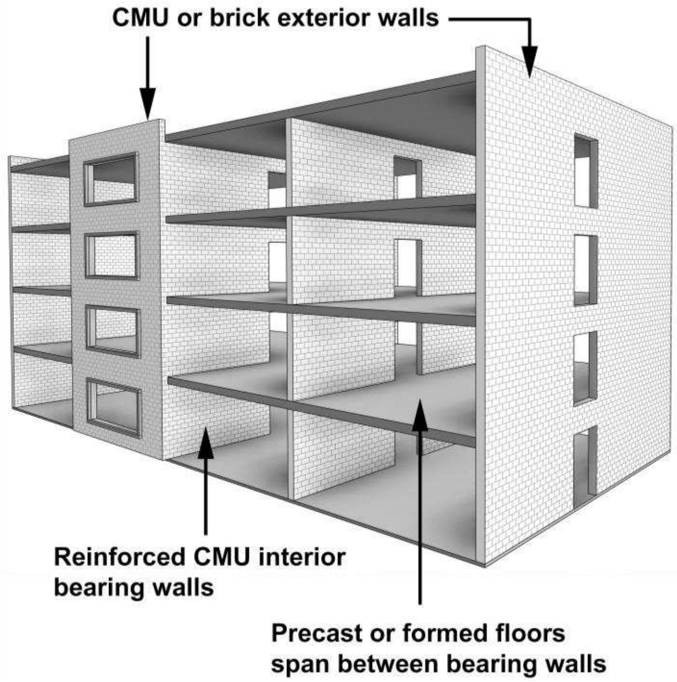

Figure B.26: Schematic illustration of a RMC *building* (Figure adapted from FEMA 547, FEMA, 2006)

Figure B.27: An example of a reinforced masonry bearing wall building (http://www.canadamasonrydesigncentre.com/about/our-staff/)

To identify reinforced masonry, one must determine separately if the building is masonry and if it is reinforced. The best way to assess reinforcement condition is to compare the construction date with the masonry reinforcement code requirement date for the local jurisdiction.

The screener also needs to determine if the building is veneered with masonry or is a masonry building. Wood siding is seldom applied over masonry. If the front façade appears to be reinforced masonry but the side has wood siding, it is probably a wood frame that has undergone façade renovation. The back of the building should be checked for signs of the original construction type.

If it can be determined that the bearing walls are constructed of concrete blocks, they may be reinforced. Load-bearing structures using these blocks are probably reinforced if the local code required it. Concrete blocks come in a variety of sizes and textures. The most common size is 190 mm wide × 390 mm long × 190 mm high. Their presence is obvious if the concrete blocks are left as the finished surface.

Typical earthquake damage

Reinforced masonry buildings can perform well in moderate earthquakes, if they are adequately reinforced and grouted, and if diaphragms are sufficiently anchored. A major problem is workmanship quality control during construction. Poor construction practices may result in ungrouted and unreinforced walls. Even where construction practice is adequate, insufficient reinforcement in the design can be responsible for extensive wall damage. The lack of positive connection of floor and roof diaphragms to walls is also a problem. Some older reinforced masonry buildings have wall-to-diaphragm tension ties that rely on cross-grain bending of the perimeter ledger to resist loads, a particularly poor detail for resisting earthquake loading.

B.5.12 **Unreinforced masonry bearing wall buildings (URM)**

Most unreinforced masonry (URM) bearing-wall structures in Western Canada and Quebec were built before the 1940s (see Figure B.28 and Figure B.29); however this construction type was also built in some jurisdictions with moderate or high seismicity until the late 1940s or late 1950s. These buildings usually range from one to six storeys in height and typically function as commercial, residential or industrial buildings. Construction varies according to type of use, although wood floor and roof diaphragms are common in older buildings. Smaller commercial and residential buildings usually have light wood floor/roof joists supported on the typical perimeter URM wall, with interior load-bearing wood partitions. Larger buildings, such as industrial warehouses, have heavier floors and interior columns, usually of wood. Bearing walls in these industrial buildings tend to be thick, often as much as 600 mm or more at the base. Wall thicknesses in residential buildings range from 230 mm on upper floors to 460 mm on lower floors.

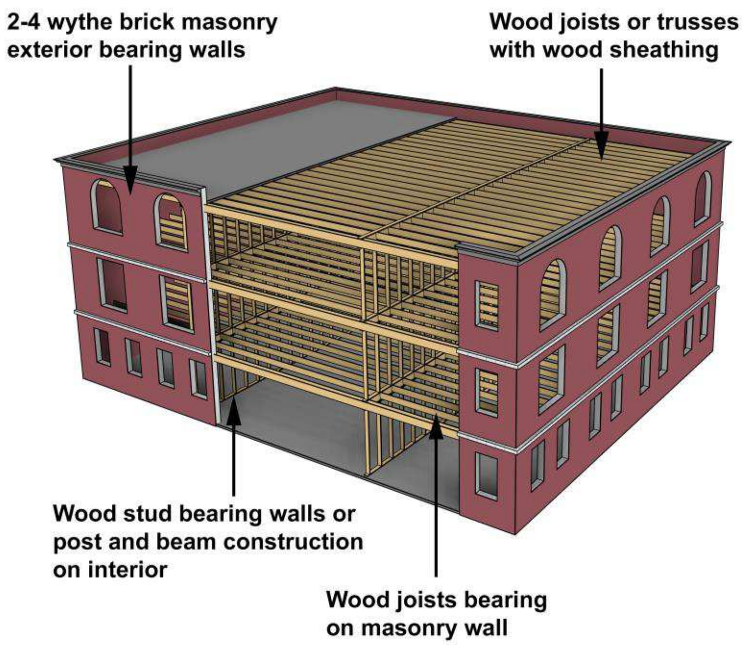

Figure B.28: Schematic illustration of a URM *building* (Figure adapted from FEMA 547, FEMA, 2006)

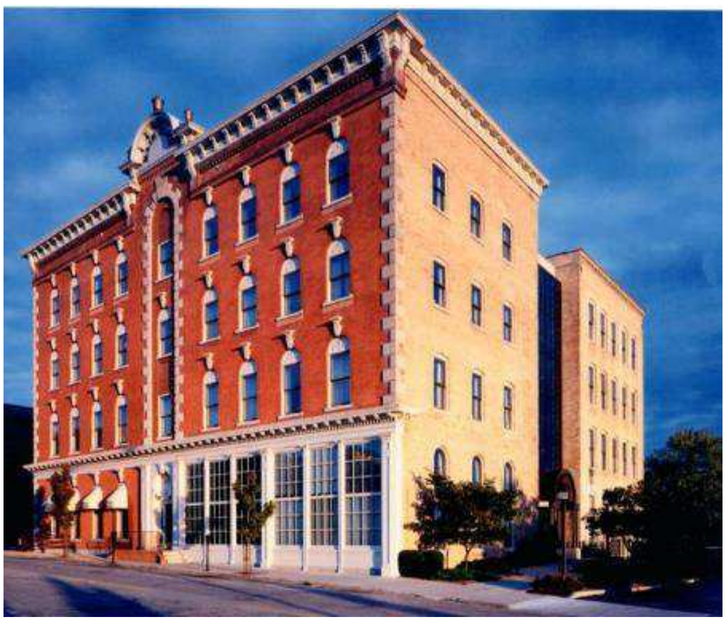

Figure B.29: Example of a URM building (http://firmaya.site/masonry-building/masonry-building-historic-glass-addition-between-historic-masonry-buildings-unreinforced-masonry-wall-design-example/)

The first step in identifying this building type is to determine if the structure has bearing walls. Second, the screener should determine the approximate age of the building. Some indications of unreinforced masonry are listed below.

- Weak mortar was used to bond masonry units together in much of the early unreinforced masonry construction in Canada. As the poor earthquake performance of this mortar type became known in the 1930s, and as cement mortar became available, this weaker mortar was no longer used and thus is not found in more recent masonry buildings. If this soft mortar is present, it is probably URM. Soft mortar can be scratched with a hard instrument such as a penknife, screwdriver or coin. This scratch testing, if permitted, should be done in a wall area where the original structural material is exposed, such as the sides or back of a building. Newer masonry may be used in renovations and it may look very much like the old. Older mortar joints can also be repointed (i.e., regular maintenance of the masonry mortar) or repaired with newer mortar during renovation. The original construction may also have used a high-quality mortar. Thus, even if the existence of soft mortar cannot be detected, it may still be URM.

- One architectural characteristic of older brick bearing wall structures is an arch or flat arch window head. This type of window head functions as a header to carry the load above the opening to either side. Although masonry-veneered wood frame structures may have these features, they are much more widely used in URM bearing wall structures, as they were the most economical method of spanning over a window opening at the time of construction. Other methods of spanning are also used, including steel and stone lintels, but these methods are generally more costly and usually only employed on the front façade.

- Some structures of this type will have anchor plates visible at the floor and roof lines, approximately 1800 mm to 3000 mm on-centre, around the perimeter of the building. Anchor plates are usually square or diamond-shaped steel plates approximately 150 mm × 150 mm, with a bolt and nut in the centre. Their presence indicates anchor ties have been placed to tie the walls to the floors and roof. These are either from the original construction or from *seismic upgrading* under local ordinances. Unless the anchors are 1800 mm on-centre or less, they are not considered effective in earthquakes. If they are closely spaced and appear to be recently installed, it indicates that the *building* has been seismically upgraded. In either case, when these anchors are present all around the *building*, the original construction is URM bearing wall.

- When a *building* has many exterior solid walls constructed from hollow clay tile and no columns of another material can be detected, it is not likely a URM bearing wall, but probably a wood or metal frame structure with URM infill.

- One way to distinguish a reinforced masonry *building* from an unreinforced masonry *building* is to examine the brick pattern closely. Reinforced masonry usually does not show header bricks in the wall surface.

If a *building* does not display the above features, or if the exterior is covered by other finish material, the *building* may still be URM.

Typical earthquake damage

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 (see Figure B.30). Typical problems include:

- **Unbraced parapets:** Parapets cantilevering up from the roof line are typically not braced and are usually the first element to fail in an earthquake (see Figure B.30).

- **Insufficient wall-to-diaphragm ties:** Because the walls, parapets, and cornices were not positively anchored to the floors, they tend to fallout. 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 *seismic upgrading*. These older anchors exhibit questionable performance.

Figure B.30: An example of parapet damage

- **Inadequate URM wall out-of-plane capacity:** URM walls that are adequately tied to the floor and roof diaphragms span between the diaphragms under out-of-plane loads perpendicular to the face of the wall. Walls with high height-to-thickness ratios are more susceptible to out-of-plane failures.

- **Inadequate URM wall in-plane capacity:** The mortar used in older URM *buildings* was often made of lime and sand, with little or no cement, and had limited shear strength. Wall lines with large openings further reduce capacity. URM bearing walls can be heavily *damaged* and collapse under large loads.

- **Inadequate diaphragm strength and stiffness:** The strength and stiffness of wood diaphragms in wood buildings can be inadequate to withstand large loads generated by heavy masonry walls. They may lack the strength to transfer out-of-plane wall loads back through the diaphragm to the in-plane walls. Displacement at mid-span of the diaphragm may amplify the out-of-plane loads on the walls and exceed interior partions' in-plane capacity. At open front situations, the diaphragm may be inadequate to transfer the load from URM perimeter walls above the open front back to other walls that can deliver the loads to the foundations.
- **Slender walls:** Some of these *buildings* have tall story 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.

B.5.13 **Manufactured homes (MH)**

Manufactured homes (MH) are prefabricated, permanently attached to a chassis, and can be relocated by being towed or transported on a trailer. "Mobile home" is an older term for a manufactured home, though it is still commonly used. Manufactured homes are built in accordance with the requirements of CSA Z240.10.1, Site preparation, foundation, and installation of *buildings*.

Manufactured homes are typically 4.3 meters wide, not more than one storey in height, and are raised off the ground by concrete blocks or metal pylons, with no anchoring to the ground. Floors and roofs are usually constructed with plywood or oriented strand board, and outside surfaces are covered with sheet metal.

Modular *buildings* are another type of manufactured homes. Modular *buildings* are prefabricated, but they do not have a permanent chassis or axles, and must be transported to the site on flatbed trucks. Modules are set on a foundation by a crane, and then joined together to create a final structure. 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 with an overall height of not exceeding three *storeys*, the NBC 1995 and its later editions require such building frames to be anchored to their foundations unless a structural analysis of wind and earthquake pressures shows anchorage is not required. Prefabricated structures are used not only as residences, but also for schools and other occupancies, as well as temporary *buildings* with many uses. Portable classrooms are often used on school properties to provide additional, temporary space.

The focus for the MH *building type* is on *buildings* that are mobile, raised off the ground, not anchored to the ground, and may or may not have an earthquake resistant bracing system (ERBS). This includes mobile homes and modular buildings such as those used for portable classrooms, when they are not permanently anchored. Note that prefabricated structures that are permanently anchored to a foundation are not considered as the MH *building type*. In such cases, the appropriate *model building types* should be chosen. For example, prefabricated structures built with a steel superstructure and anchored to a foundation are considered to be steel moment frame (SMF) *building type*.

In the *Level 2 – SQST*, it may not always be possible to determine whether a permanent foundation or an ERBS exists, as there is often a cripple wall or skirt wall covering the underlying conditions. Unanchored manufactured homes should be assumed unless a permanent foundation or ERBS can be seen.

Typical earthquake damage

Numerous studies have shown that manufactured homes performed significantly worse than conventional wood-frame dwellings in the 1994 Northridge earthquake (SSC 1995). Typical problems are summarized as follows:

- The main weakness in manufactured homes during seismic activity is related to their shifting from their foundations.
- Cracking of partitions and ceilings and cracking of walls.
- In moderate shaking, the building can fall off its supports, causing jack stands to penetrate the floor.
- Connected utility lines can be severed, and escaping gas can cause fires.
- Falling objects, structural damage and fire can lead to injuries and fatalities.

APPENDIX C FEMA P-154 SCORING METHODOLOGY

FEMA P-154 (2015) provided a quantitative methodology that evaluates the *seismic risk* of *existing buildings* based on the probability of *building* collapse. The *building* collapse in this context means that any part of the gravity system experiences dynamic instability leading to the loss of load bearing capacity. In this Appendix, the FEMA P-154 methodology is presented in detail.

C.1 An overview of the FEMA P-154 methodology

In FEMA P-2015, the *seismic risk* of an *existing building* is represented by structural score *S*, which is defined as the negative common logarithm of the probability of collapse for a given risk-targeted maximum considered earthquake (MCE_R) ground shaking, i.e.:

$$S = -\log_{10}[P(COL|MCE_R)] \tag{C.1}$$

Conversely, $P(COL|MCE_R)$ can be obtained as follows:

$$P(COL|MCE_R) = 10^{-S} \tag{C.2}$$

The scoring procedure begins by identifying the *seismicity* region, Soil Type and *building* characteristics, including *building type*, *building irregularity*, *building age*, etc. Since it is not feasible to score every individual *building*, the *building* attributes are generalized for seventeen common *building types*. The final score *S* is calculated based on the following:

$$S = S_B + \sum_i M_i \geq S_{\min} \tag{C.3}$$

where S_B represents the basic score, M_i represents applicable score modifiers and S_{min} represents the minimum score. The S_B value is determined for a typical *building type* that was designed and built between the *pre-code* year (the year when seismic code requirements were first adopted and enforced) and *benchmark year* (the year when major improvements in seismic code requirements were adopted and enforced), without any deficiencies (such as *irregularities* and *deterioration*), and is located in Soil Type CD (average of Soil Types C and D), and corresponding to a specified *seismicity* region (e.g. Low *seismicity* region). The M_i accounts for the presence of *building* conditions that affect a *building's* seismic performance, such as Soil Type, *building irregularity*, *pre-code building*, and *post-benchmark building*. It is noticed that *S* is determined by adding one or more applicable score modifiers to the basic score, ignoring the possible interconnections among the conditions and thus results, for some extreme cases, in negative final scores, indicating that the probability of collapse is greater than 1. However, the probability of being greater than 1 has no physical meaning. To avoid a possible negative final score, a minimum score $S_{\min}$, representing the *building's* worst condition, is calculated by assuming all *building* deficiencies occur at once. $S_{\min}$ is always positive regardless of *seismicity* region or *building type*.

The final score S is compared against a cut-off score of 2.0 to assess the relative seismic vulnerability of the *building*. A *building* with a final structural score $S \geq 2$ is exempt from structural seismic evaluation; otherwise, detailed structural seismic evaluation is required.

C.2 Determination of the basic score

Given a *model building type* and *seismicity* region, the basic score S_B is calculated as follows:

$$S_B = -\,log_{10}[P(COL|MCE_R)] \tag{C.4}$$

where $P(COL|MCE_R)$ is the probability of collapse given an MCE_R corresponding the specified *seismicity* region and is calculated by:

$$P(COL|MCE_R) = P(COL|Complete\ damage) \times P\big(Complete\ damage|\delta_p\big) \tag{C.5}$$

where $P(COL|Complete\ Damage)$ equals the empirical collapse factor CF, which represents the expected collapsed area of the *building*, and $\big(Complete\ Damage|\delta_p\big)$ is the probability of the *building* being in complete *damage* state given peak *building* response δ_p.

$P(COL|MCE_R)$ is calculated based on the *building* capacity and fragility curves for a specified *model building type* and *seismicity* region.

C.3 Development of building capacity curves

The *building* capacity curve is a plot of the base shear and roof displacement from a nonlinear pushover curve that has been converted to acceleration-displacement response spectrum (ADRS) format. The shape of the capacity curve should be an accurate approximation of the *building's* lateral load resistance and ductility. Although a *building's* seismic performance and potential *damage* vary based on the type of structural system used, the type of construction material (i.e., wood, steel, concrete, masonry) and type of SFRS (i.e., moment frame, shear wall, etc.) are two important factors that influence a *building's* seismic performance. Since it is not feasible to construct a capacity curve for every conceivable *building*, screening tools commonly generalize *building type*s according to common materials and SFRS systems, known as *model building types*. The capacity curve for each *model building type* is constructed using *building*-specific properties that describe the median behaviour expected of a particular *model building type* (FEMA 2003a).

Building capacity curves are described using two control points, one for yield capacity (D_y, A_y) and another for ultimate capacity (D_u, A_u), as shown in Figure C.1 and given in Eqns. (C.6) to (C.9).

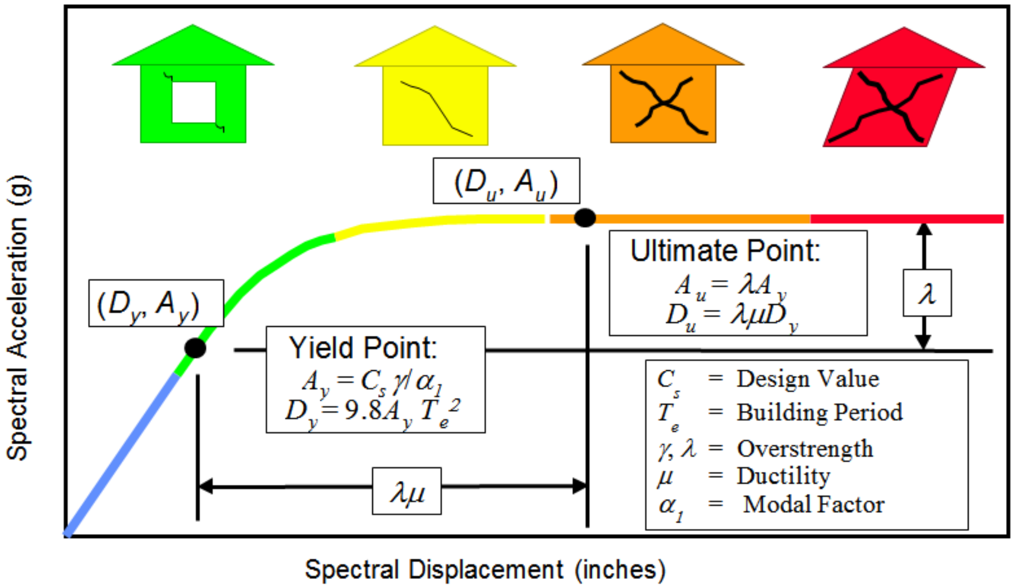

Figure C.1: Example of a building capacity curve and controlling points (picture from Figure 4-3 in FEMA P-155 2015)

$$A_y = \frac{C_s \gamma}{\alpha_1} \tag{C.6}$$

$$D_y = 9.8 A_y T_e^2 \tag{C.7}$$

$$A_u = \lambda A_y \tag{C.8}$$

$$D_u = \lambda \mu D_y \tag{C.9}$$

where: C_s is the seismic design coefficient, which is a ratio of the *building* weight to seismic base shear, based on design code level. C_s is approximately based on the lateral-force design requirements of current seismic codes (e.g., 1994 NEHRP Provisions). These requirements are a function of the *building's* location and other factors including: soil condition, type of SFRS and the *building's* fundamental period. α_1 is the modal weight factor, which is the fraction of *building* weight participating in the pushover mode. T_e is the *building's* fundamental period. γ is the yield strength factor, which is the ratio of nominal yield strength to design strength. λ is the ultimate strength factor, which is the ratio of nominal ultimate strength to yield strength. μ is the ductility factor, which is the ratio of ultimate displacement to λ times the yield displacement. The specific values of the above parameters for seventeen *model building types* are presented in Appendix A of FEMA P-155 (2015).

The entire capacity curve can be constructed once the *building* control points have been established. The *building* is assumed to be perfectly elastic prior to yielding, and perfectly plastic beyond the ultimate capacity. An elliptical shape that is tangent to the elastic segment at yield capacity point and tangent to the plastic segment at the ultimate capacity point is used to describe the capacity curve between the yield and ultimate control points as follows (Tokas and Lobo 2009):

$$A = b\sqrt{1 - \frac{(D - D_u)^2}{a^2}} + k \tag{C.10}$$

where the coefficients a, b, and k are given as:

$$a = b\sqrt{\frac{D_y}{A_y}\frac{\left(D_u - D_y\right)}{\left(A_y - k\right)}} \tag{C.11}$$

$$b = A_u - k \tag{C.12}$$

$$k = \frac{A_u^2 - A_y^2 + \frac{A_y^2}{D_y}\left(D_y - D_u\right)}{2\left(A_u - A_y\right) + \frac{A_y}{D_y}\left(D_y - D_u\right)} \tag{C.13}$$

The actual response corresponding to the red portion of the capacity curve in Figure C.1 does not remain flat given the post-peak loss of strength. For analysis purposes however, HAZUS AEBM (2003a) assumes the structure maintains constant strength beyond the complete *damage* state.

C.4 Construction of seismic demand spectrum

The seismic demand spectrum, namely inelastic response spectrum, is determined based on the 5%-damped design response spectrum reduced to account for viscoelastic damping inherent to the *building type* and energy dissipation due to nonlinear response and hysteretic behaviour. Like the capacity curve, the seismic demand spectrum is defined in the ADRS format with spectral response acceleration $S_a(T)$ on the vertical axis and spectral displacement $S_d(T)$ on the horizontal axis.

In FEMA P-154, the design response spectrum is determined based on spectral acceleration at 0.2-second period S_{MS}, and spectral acceleration at 1-second period S_{M1}, corresponding to the MCE_R associated with a specified *seismicity* region. Although the actual design response spectrum depends on whether the earthquake occurs in western North America or central and eastern North America, whether it is a large or moderate size event and whether the site is near or far from the earthquake source, the differences between the shape of an actual spectrum and the standard spectrum tend to be significant only at periods less than the short period, and at periods greater than T_{VD} with a default value of 10 seconds, which do not significantly affect the accuracy of the HAZUS methodology (HAZUS TM 2003b), which is the rational basis of FEMA P-154.

Because Soil Type CD is chosen as the reference Soil Type, S_{MS} and S_{M1} corresponding to Soil Type CD are taken to develop the elastic response spectrum. Equations to determine spectral response accelerations of the elastic response spectrum with damping ratios other than 5% are presented as follows:

$$S_a(T) = S_{MS}/R_A \qquad \text{for } 0 \leq T \leq T_s \tag{C.14}$$

$$S_a(T) = (S_{M1}/T\,)/R_V \qquad \text{for } T_S < T \leq T_{VD} \tag{C.15}$$

$$T_S = \left[\frac{S_{M1}}{S_{MS}}\right] \cdot \left(\frac{R_A}{R_V}\right) \tag{C.16}$$

$$T_{VD} = 10^{\frac{M_w - 5}{2}} \tag{C.17}$$

where R_A and R_V are damping correction factors given by

$$R_A = \frac{2.12}{3.21 - 0.68 \ln \beta_e} \tag{C.18}$$

$$R_V = \frac{1.65}{2.31 - 0.41 \ln \beta_e} \tag{C.19}$$

in which β_e is elastic damping ratio of the *building*. When $\beta_e = 5\%$, $R_A = 1.0$ and $R_V = 1.0$, no modification is needed. Elastic damping β_e values are summarized in Appendix A of FEMA P-155. HAZUS TM (2003b) recommends that T_{VD} in Eq. (C.17) be estimated using the relationship proposed by Joyner and Boore (1988) to predict the moment magnitude M_w of an earthquake. When M_w is unknown, the period T_{VD} is assumed to be 10 seconds (i.e., moment magnitude assumed to be $M_w = 7$) (FEMA 2003b).

HAZUS TM used the following equation to determine spectral displacement given spectral response acceleration at a specific period:

$$S_d(T) = 9.8 \cdot S_a(T) \cdot T^2 \tag{C.20}$$

To construct the seismic demand spectrum, an effective damping β_{eff} instead of β_e is used in Eqns. (C.18) and (C.19) to determine damping correction factors (Newmark and Hall 1982; FEMA P-155 2015). β_{eff} is determined by the following Eq. (C.21):

$$\beta_{eff} = \beta_e + \beta_H \tag{C.21}$$

Since the hysteretic damping β_H is amplitude-dependent, greater levels of inelastic displacement correspond to greater potential for *building* degradation and energy dissipation under seismic loading. The hysteretic damping β_H can be calculated as follows:

$$\beta_H = \kappa_d \left(\frac{Area}{2\pi DA}\right) \tag{C.22}$$

where D is the peak spectral displacement of the system (i.e., S_d in Eq. (C.20)), A is the spectral response acceleration at peak spectral displacement (i.e., S_a in Eqns. (C.14) and (C.15)), $Area$ is the area enclosed by an idealized hysteretic loop constructed between the positive and negative peak spectral displacements $\pm S_d$, and κ_d is a degradation factor that defines the amount of hysteretic damping as a function of earthquake duration. The κ factor is used to represent how well the actual *building* hysteresis is represented by the idealized hysteresis curve used to calculate $Area$ (ATC 1996). κ_d values are provided in Appendix A of FEMA P-155, and range from 0.3 to as high as 0.668, depending on relative quality of seismic construction and the level of earthquake hazard.

The hysteresis area used to calculate the β_{eff} is influenced by a number of factors, including the type and material used to construct the SFRS, the magnitude and duration of ground shaking, and the magnitude of inelastic *building* displacement. The method proposed by Tokas and Lobo (2009) was used to calculate the hysteretic area. Compared to ATC (1996) that used bilinear hysteretic loop model, Tokas and Lobo used a more accurate hysteretic loop model to calculate the hysteretic area. Nevertheless, the formulas to calculate effective damping and inelastic seismic demand remain unchanged. The Tokas and Lobo model for hysteretic area is based on three simplifying assumptions: (i) unloading stiffness, which is the same as initial elastic stiffness; (ii) onset of the transition zone in the unloading branch, which occurs at the same spectral response acceleration as the yield points on the backbone capacity curve, and; (iii) strength degradation, and pinching of the hysteretic curve, which is ignored. The assumed hysteretic loop is shown in Figure C.2. The calculation of hysteretic area is dependent on whether *building* displacement S_d is in the elastic region ($0 < S_d \le D_y$), elastic-plastic transition phase ($D_y < S_d \le D_u$), or in the post-yield plateau ($S_d > D_u$).

For the response in the elastic region, $0 < S_d \le D_y$:

$$\begin{aligned} S_a &= \left(\frac{A_y}{D_y}\right) S_d \\ A_{inc} &= 0 \\ Area &= 0 \end{aligned} \tag{C.23}$$

For the response in the elastic-plastic transition phase, $D_y < S_d \le D_u$:

$$S_a = b\sqrt{1 - \frac{(S_d - D_u)^2}{a^2}} + k \tag{C.24}$$

$$A_{inc} = S_a \cdot (S_d - D_y)$$
$$- \text{Area under the curve from } D_y \text{ to } S_d \text{ by numerical integration}$$

$$Area = 4S_aS_d - (S_a + A_y)^2 \left(\frac{D_y}{A_y}\right) - 2(S_a^2 - A_y^2)\left(\frac{D_y}{A_y}\right) - 2A_{inc}$$

where a, b ,and k are obtained from Eqns. (C.11) through (C.13).

For response in the post-yield plateau, $S_d > D_u$

$$S_a = A_u$$

$$A_{inc} = \left(\frac{4-\pi}{4}\right)(S_a - A_y)(D_u - D_y) \tag{C.25}$$

$$Area = 4S_aS_d - (S_a + A_y)^2 \left(\frac{D_y}{A_y}\right) - 2(S_a^2 - A_y^2)\left(\frac{D_y}{A_y}\right) - 2A_{inc}$$

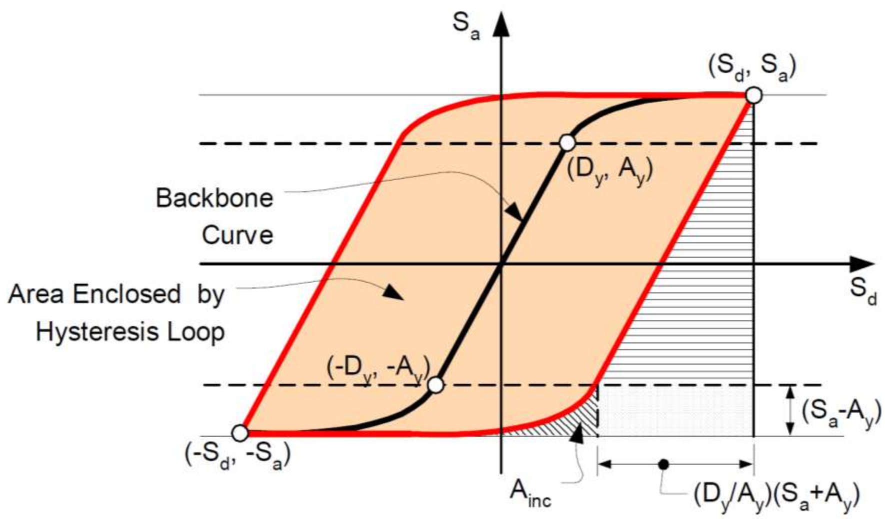

Figure C.2: Idealized hysteretic loop used to calculate hysteretic damping β_H (from Tokas and Lobo 2009)

C.5 Calculation of peak spectral response

FEMA P-154 adopted the capacity-spectrum method (CSM) to determine peak spectral response δ_p, also known as the performance point. By definition, this point is the intersection of the capacity curve and the locus demand spectrum, implying a sense of dynamic equilibrium (FEMA 440 2005). The spectral displacement corresponding to the performance point is the expected *building* displacement given the *MCE$_R$* corresponding to a specified *seismicity* region. An iterative procedure is required to find the performance point, since the reduced seismic demand spectrum is based on the equivalent period T and effective damping β_{eff}, which are functions of the structures' spectral displacement S_d. FEMA P-154 (2015) used a graphical solution to obtain δ_{pi} by "calculating the period and effective damping at each possible displacement, and then plotting spectral displacement versus spectral response acceleration for each value of displacement as effective damping and period vary." The β_{eff}-damped response spectrum can be constructed by stepping along the capacity curve for incremental values of displacement D_i, and performing the following calculations:

1. Select an initial incremental point D_i, A_i on the capacity curve;
2. Compute the incremental period $T_i = \sqrt{D_i/9.8A_i}$;
3. Compute the hysteretic damping β_H (Eq. (C.22)) by using Eqns. (C.23) to (C.25) to obtain the area enclosed by the hysteretic loop;
4. Calculate the effective damping β_{eff} (Eq. (C.21));
5. Calculate the damping reduction factors R_A (Eq. (C.18)) and R_v (Eq. (C.19));
6. Determine the incremental β_{eff}-damped response spectra $S_a(T_i)$ (Eq. (C.14) for $T_i \leq T_s$, Eq. (C.15) for $T_s < T_i \leq T_{VD}$) and $S_{d,i}$ (Eq. (C.20));
7. If $S_{d,i} < D_e$, then $S_{d,i} = D_e$, where the elastic displacement D_e can be calculated from Eq. (C.20) using elastic period T_e;
8. Repeat for a larger incremental displacement and return to Step 1.

By performing the iterative procedure above, one can obtain the performance point δ_p, as shown in Figure C.3.

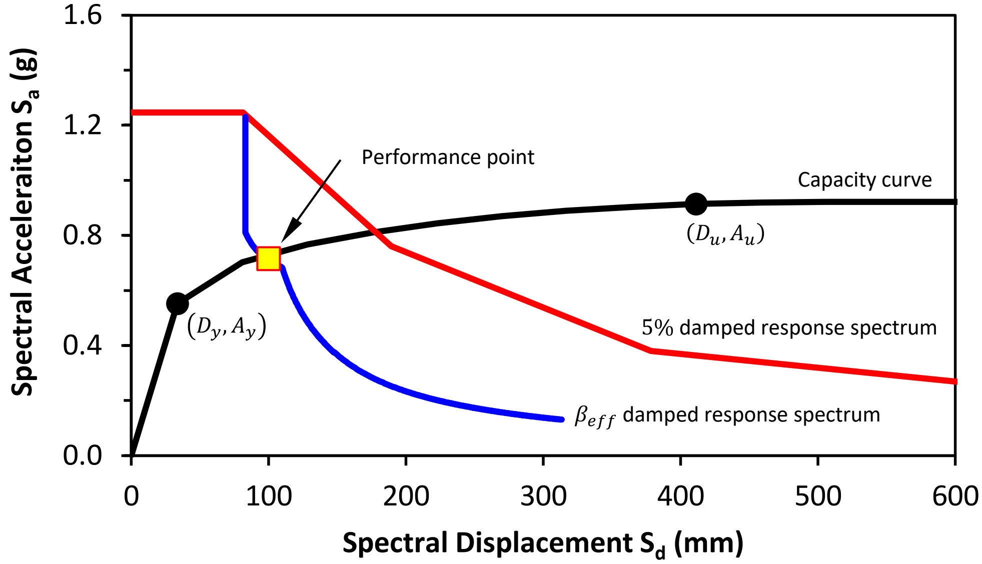

Figure C.3: Capacity-spectrum method for determining peak spectral response

C.6 Development of fragility curves

Building fragility curves are lognormal functions that describe the probability of reaching or exceeding structural and non-structural *damage* states, given median estimates of spectral response such as spectral displacement. These curves take into account the variability and uncertainty associated with capacity curve properties, *damage* states and ground shaking intensity.

HAZUS AEBM (20012a) developed fragility curves with respect to four *damage* states, i.e., slight, moderate, extensive, and complete *damage* states for typical *model building types* assigned to various design code levels. Figure C.4 provides an example of HAZUS fragility curves for slight, moderate, extensive, and complete structural *damage* states, respectively, and illustrates differences in *damage* state probabilities for three levels of spectral response corresponding to weak, medium, and strong earthquake ground shaking, respectively. The terms "weak," "medium," and "strong" are used here for simplicity; in the actual methodology, only quantitative values of spectral response are used.

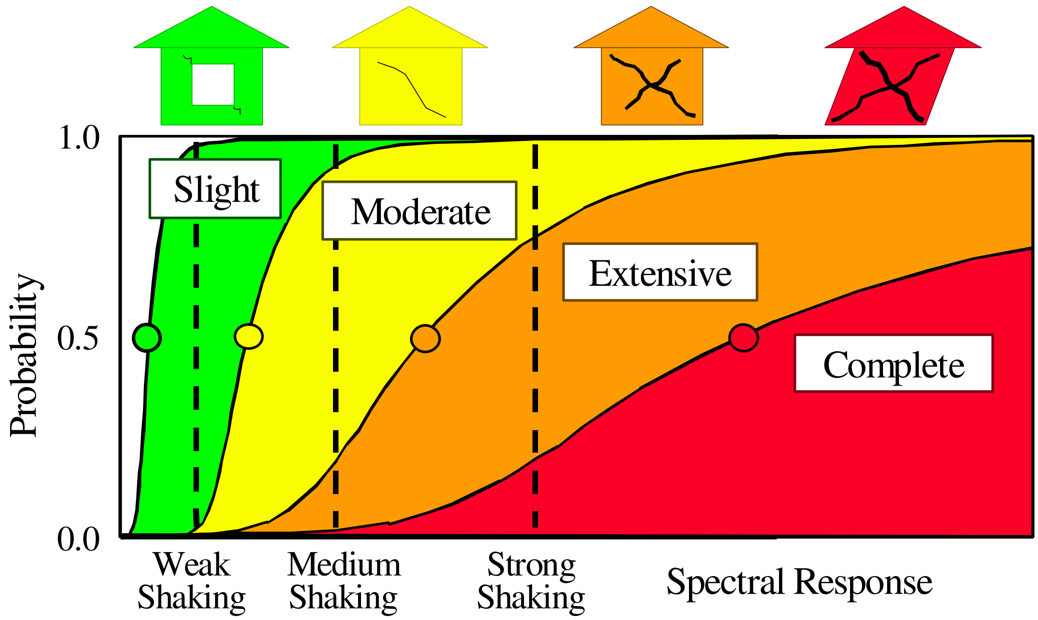

Figure C.4: An example of HAZUS fragility curves for slight, moderate, extensive, and complete *damage* states (from FEMA P-155 2015)

Each fragility curve is defined by a median value of the demand parameter (e.g., spectral displacement) that corresponds to the threshold of that *damage* state, and by the variability associated with that *damage* state. Median values of demand parameter are based on a number of published sources (OAK 1994; Kustu et al. 1982; Ferritto 1982, 1983; Czarnecki 1973; Hasselman et al. 1977; Wong 1975). Variability is modeled by combining three contributors to structural *damage* variability, including the variability of capacity curve, variability of seismic demand spectrum, and uncertainty in the estimated median value of the structural *damage* state threshold. Generic standard deviation values are assigned to these three types of variability (Tables 6.5 to 6.7 in HAZUS AEBM 2012a). Given values of median interstorey drift ratios, modal factors, and standard deviations, fragility curves for typical *building types* (e.g., steel moment frame), with respect to a specified design code level (e.g., Moderate-code design level), can be developed.

While HAZUS AEBM (2012a) defines various *damage* states spanning from slight *damage* state to complete *damage* state, the collapse of a *building* is most closely related to complete *damage* state. Given any value of spectral response, e.g., spectral displacement S_d for the structure, probability of a structure being in a specified *damage* state is determined by:

$$P[Complete\ damage|S_d] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{S_d}{S_{d,m}}\right)\right] \tag{C.26}$$

where $S_{d,m}$ is the median value of the spectral displacement corresponding to the threshold of that *damage* state and β_d is the logarithmic standard deviation of spectral response variability associated with that *damage* state.

The median value of spectral displacement $S_{d,m}$, corresponding to structural complete *damage* state is calculated as:

$$S_{d,m} = \Delta_c \, H_R \, (\alpha_2/\alpha_3) \tag{C.27}$$

where Δ_c is the interstorey drift ratio corresponding to structural complete *damage* state, H_R is the *building* height at roof level in inches, and α_2 and α_3 are the modal height factor and modal shape factor relating maximum *storey* drift and roof drift. Specific values of these parameters are provided in Appendix A of FEMA P-155 (2015).

Standard deviation β_d associated with the structural complete *damage* state is determined by:

$$\beta_d = \sqrt{\frac{\beta_{C,D}^2 - (\beta_{CAP}^2 + \beta_{T,C}^2)}{X\left(1 + \frac{D_e}{D}\right)} + (\beta_{CAP}^2 + \beta_{T,C}^2)} \tag{C.28}$$

where $\beta_{C,D}$ represents standard deviation that is deterministically based on HAZUS AEBM (2012a) and OSHPD HAZUS (2010), which depends on standard deviation β_{CAP} of capacity variability and $\beta_{T,C}$ of *damage* state threshold variability. Since probabilistic ground motions *MCE$_R$* are used, deterministic $\beta_{C,D}$ is reduced by Eq. (C.28) to avoid double counting the variability associated with demand and final standard deviation β_d, which is used to develop fragility curve (FEMA P-155 2015). D_e and D are the elastic deformation and peak spectral displacement, respectively. X is a factor used to adjust the reduction of $\beta_{C,D}$.

In HAZUS AEBM (2012a), $\beta_{C,D}$ values are functions of degradation factors κ_d, β_{CAP} and $\beta_{T,C}$, as well as *building* height H_R. Generally, β_{CAP} ranges between 0.1 and 0.4, $\beta_{T,C}$ ranges between 0.2 and 0.6, and H_R is divided into low-, mid-, and high-rise. The β_{CAP} depends on ultimate strength uncertainty, while $\beta_{T,C}$ depends on construction quality, population of *buildings*, and knowledge of seismic responses. For example, HAZUS TM (2012b) uses $\beta_{CAP} = 0.25$ for ordinary code *buildings* and $\beta_{CAP} = 0.3$ for ordinary *pre-code buildings*. Similarly, a smaller $\beta_{T,C}$ is expected for individual low-rise *buildings* with well-known properties, as compared to higher *buildings* with larger uncertainty in seismic performance. For generic *model building types* that represent large populations of *buildings* for which properties are not well known, $\beta_{CAP} = 0.3$ and $\beta_{T,C} = 0.3$ are used in the original development of HAZUS fragility curves (HAZUS TM 2012b).

OSHPD HAZUS (2010) presumably adopted $\beta_{CAP} = 0.3$ and $\beta_{T,C} = 0.3$ to determine that $\beta_{C,D}$=0.9 of baseline performance in pre-1961 low-rise structures (1-3 *storey*); however, it did not follow Tables 6.6 and 6.7 in HAZUS AEBM for mid- and high-rise *buildings*, but set a lower bound baseline $\beta_{C,D} = 0.8$ ($\beta_{C,D} = 0.75$ in Table 6.7) for pre-1961 high-rise *buildings* and applied linear reduction (-0.01) for 4- to 13-*storey buildings*. For baseline performance in post-1961 low-rise *buildings*, $\beta_{CAP} = 0.2$ and $\beta_{T,C} = 0.2$ are used, taking into consideration the benefits of improved seismic design requirements and construction quality; for sub-base performance of low-rise *buildings*, $\beta_{CAP} = 0.3$ and $\beta_{T,C} = 0.4$ are taken for post-1961, while $\beta_{CAP} = 0.4$ and $\beta_{T,C} = 0.4$ are adopted for pre-1961 *buildings*, addressing the detrimental effect of deficiencies. For mid- and high-rise *buildings*, the same method used for calculating $\beta_{C,D}$ of *buildings'* baseline performance is used.

In FEMA P-154 (2015), $\beta_{CAP} = 0.3$ and a higher $\beta_{T,C} = 0.4$ are used, resulting in a larger $\beta_{C,D}$=0.95 for basic score calculations of low-rise *buildings*, compared to $\beta_{C,D}$=0.9 of the baseline performance of pre-1961 *buildings* in OSHPD HAZUS (2010). It is noticed that, in HAZUS TM (2012b), $\beta_{CAP} = 0.25$ for ordinary code *buildings*, $\beta_{CAP} = 0.3$ for ordinary *pre-code buildings*, and $\beta_{T,C} = 0.4$ for all ordinary *buildings*. FEMA P-154 seems to simplify the determination of β_{CAP} (i.e., $\beta_{CAP} = 0.3$) with negligible effect on the final $\beta_{C,D}$. For mid- and high-rise *buildings*, FEMA P-154 followed the same method used in OSHPD HAZUS. A mapping of $\beta_{s,C}$ between FEMA P-154 and OSHPD HAZUS is presented in Appendix A of FEMA P-155. $X = 0.75$ is used in all cases.

Having determined $S_{d,m}$ and β_d, the probability of the structure being in complete *damage* state given spectral displacement S_d can be determined by:

$$P[Complete\ damage|S_d] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{S_d}{S_{d,m}}\right)\right] \tag{C.29}$$

Changing the S_d values in the equation above from a small displacement such as $D_u/500$ to a large displacement S_d>D_u results in a fragility curve.

Given peak *building* response δ_p determined in Section C.5, the probability of a structure being in complete *damage* state is determined by:

$$P\left[Complete\ damage\middle|\delta_p\right] = \Phi\left[\frac{1}{\beta_d}\ln\left(\frac{\delta_p}{S_{d,m}}\right)\right] \tag{C.30}$$

C.7 Calculation of the basic score

It is noted that *building* collapse differs from complete *damage* state. The latter may or may not include any collapse in any given portion. Similar to HAZUS AEBM (2012a), an empirical collapse factor CF is used to associate the probability of structural complete *damage* state to probability of collapse, as follows:

$$P(COL|MCE_R) = P\left[Complete\ damage\middle|\delta_p\right] \times CF \tag{C.31}$$

where CF is dependent on *model building types* and the type of *irregularity*. CF values are provided in Appendix A of FEMA P-155.

Having obtained probability of collapse $P(COL|MCE_R)$, basic score S_B is computed as:

$$S_B = -\log_{10}\big(P(COL|MCE_R)\big) \quad \text{(C.32)}$$

C.8 Determination of score modifiers

The "default" parameter values used in HAZUS AEBM (2012a) were developed for typical *model building types* and do not apply to specific *buildings*. In addition, HAZUS fragility functions are appropriate for fairly regular *buildings*, but tend to underestimate *damage* in *buildings* with deficiencies. To address this issue, OSHPD HAZUS (2010) recommended adjusting the default parameter values in HAZUS AEBM appropriately in order to account for *building* deficiencies. OSHPD HAZUS developed a deficiency matrix to present potential deficiencies and their relative effect on *building* capacity, fragility and collapse factor. For example, soft *storey irregularity* could affect a *building's* maximum inter*storey* drift ratio, model shape factor and collapse factor. A set of tables are included in OSHPD HAZUS, which provide adjusted parameter values with regard to corresponding deficiencies.

FEMA P-154 adopted the score procedure developed by OSHPD HAZUS (2010) and modified a few parameter values based on the HAZUS TM (2012b) and engineering judgement. Because FEMA P-154 used *seismicity* regions to group locations that differ from *seismic zones* used in OSHPD HAZUS, a mapping between *seismicity* regions and *seismic zones* was established.

The probability of collapse given the MCE_R corresponding to a specified *seismicity* and a specific condition i, $P[COL|MCE_R, Condition\ i]$ is calculated using the adjusted parameter values. A modified structural score is determined by applying the negative common logarithm of $P[COL|MCE_R, Condition\ i]$, i.e., $S_{Condition\ i} = -\log_{10}(P[COL|MCE_R, Condition\ i])$. Score modifier M_i is calculated by subtracting the S_B from the modified structural score, i.e.:

$$M_i = S_{Condition\ i} - S_B \quad \text{(C.33)}$$

A positive M_i value indicates that condition i is beneficial to the *building's* seismic performance. In contrast, a negative M_i value indicates that condition i has detrimental effect on the *building's* seismic performance.

Compared to OSHPD HAZUS (2010), FEMA P-154 considers potential beneficial conditions including *redundancy*, *post-benchmark* and retrofit as well as *pounding,* which would result in detrimental effect on the *building* seismic performance. A total of eight *building* conditions (beneficial or detrimental) are considered in FEMA P-154. In particular, vertical *irregularity*, plan *irregularity*, *pre-code*, *post-benchmark*, and *Soil Type* are addressed in Level 1 screening, while *redundancy*, *pounding* and retrofit are addressed in Level 2 optional screening. Note that Level 1 score modifiers are determined by calculating $P[COL|MCE_R, condition\ i]$ based on adjusted parameter values; however, Level 2 score modifiers including *redundancy*, *pounding*, and retrofit modifiers are obtained from combining the relative severity of the conditions and Level 1 score modifiers based on engineering judgement. For example, the severity of *redundancy* on the *building's* seismic performance is considered to be low; therefore, the *redundancy* score modifier is set as equal to one-third of the average of *severe vertical irregularity* score modifiers for all FEMA P-154 *building types*.

C.9 Minimum score

Final score S is calculated as the sum of basic score S_B and one or more applicable score modifiers M_i. This method is an approximate approach disregarding the potential interconnections among deficiencies.

A minimum score S_{min} is developed for each *model building type* in each *seismicity* region by considering that all deficiencies in Level 1 screening, namely *severe vertical irregularity*, plan *irregularity*, *pre-code building*, and Soil Type E, occur at once. Values of S_{min} are positive for all FEMA *building types* and all *seismicity* regions.

C.10 Use of seismic screening results

Final score S is compared to a cut-off score to assess the *building's* relative seismic vulnerability. Note that the average score for new ordinary *buildings* (equivalent to Risk Category II) is approximately equal to 2.5 (FEMA P-155 2015). Assuming *existing buildings* have a somewhat lower S value than new *buildings*, FEMA P-154 selects an acceptable collapse probability P of 1 in 100 (1%), which corresponds to a cut-off score of 2.0. Therefore, a *building* with a final score $S < 2.0$ would trigger detailed seismic evaluation, as the level of ground shaking is associated with a collapse probability greater than 1%. Applications of the FEMA P-154 scoring procedure to Risk Categories III and IV, such as hospitals, were considered. However, the focus of the screening is to identify the risk to *life safety* due to collapse. In other words, FEMA P-154 cannot be used to evaluate the *seismic risk* of these *buildings* where higher performance objectives, such as continued functionality in a hospital, are desired.

APPENDIX D BUILDING-SPECIFIC PROPERTIES

Table D.1: Building specific properties for capacity curve development

MBT		No. Storeys	H_R (ft)	T_e (s)	Basic score, C_s				Pre-code modifier, C_s				Post-benchmark modifier, C_s				Gamma, γ	Lambda, λ		Ductility, μ		Modal factors	
NRC	FEMA				VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX		Basic Score	Pre-code	Basic Score	Post-benchmark	α_1	α_2
WLF	W1	1	14	0.35	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2.7	2	1.5	6	7.98	0.8	0.75
WLF	W1	2	24	0.38	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2.5	2	1.5	6	7.98	0.8	0.75
WLF	W1	3	34	0.49	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2.25	2	1.5	4.94	6.57	0.8	0.75
WLF	W1	4	44	0.60	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2	2	1.5	4.41	5.87	0.8	0.75
WLF	W1	5	54	0.70	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	1.88	2	1.5	4.07	5.41	0.8	0.75
WLF	W1	6	64	0.80†	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	1.8	2	1.5	3.82	5.08	0.79	0.72
WPB	W2	1	14	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	2	1.5	6	7.98	0.8	0.75
WPB	W2	2	24	0.38	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	2	1.5	6	7.98	0.8	0.75
WPB	W2	3	34	0.49	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	2	1.5	4.94	6.57	0.8	0.75
WPB	W2	4	44	0.60	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	2	1.5	4.41	5.87	0.8	0.75
WPB	W2	5	54	0.70	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	2	1.5	4.07	5.41	0.8	0.75
WPB	W2	6	64	0.80†	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	2	1.4	3.82	5.08	0.79	0.72
SMF	S1	1	14	0.4	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	2	1.5	6	7.98	0.75	0.75
SMF	S1	2	24	0.5	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	2	1.5	6	7.98	0.75	0.75
SMF	S1	3	36	0.69	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	2	1.5	4.94	6.57	0.75	0.75
SMF	S1	4	48	0.87	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	2	1.5	4.41	5.87	0.75	0.75
SMF	S1	5	60	1.04	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	2	1.5	4.07	5.41	0.75	0.75
SMF	S1	6	72	1.2	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	2	1.5	3.82	5.08	0.73	0.72
SMF	S1	7	84	1.36	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	2	1.5	3.63	4.83	0.71	0.69
SMF	S1	8	96	1.51	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	2	1.5	3.48	4.63	0.69	0.66
SMF	S1	9	108	1.66	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	2	1.5	3.35	4.46	0.67	0.63
SMF	S1	10	120	1.81	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	2	1.5	3.24	4.31	0.65	0.6
SMF	S1	11	132	1.95	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	2	1.5	3.15	4.19	0.65	0.6
SMF	S1	12	144	2.09	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	2	1.5	3.07	4.08	0.65	0.6
SMF	S1	13	156	2.23	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.034	0.064	0.102	1.65	2	1.5	3	3.99	0.65	0.6
SMF	S1	14	168	2.36	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	2	1.5	3	3.99	0.65	0.6
SMF	S1	15	180	2.5	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	2	1.5	3	3.99	0.65	0.6
SBF	S2	1	14	0.4	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.67	1.33	6	7.98	0.8	0.75
SBF	S2	2	24	0.43	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.67	1.33	6	7.98	0.8	0.75
SBF	S2	3	36	0.59	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.67	1.33	4.94	6.57	0.8	0.75
SBF	S2	4	48	0.73	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	1.67	1.33	4.41	5.87	0.8	0.75
SBF	S2	5	60	0.86	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	1.67	1.33	4.07	5.41	0.8	0.75
SBF	S2	6	72	0.99	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	1.67	1.33	3.82	5.08	0.79	0.72
SBF	S2	7	84	1.11	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	1.67	1.33	3.63	4.83	0.78	0.69
SBF	S2	8	96	1.22	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	1.67	1.33	3.48	4.63	0.77	0.66
SBF	S2	9	108	1.34	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	1.67	1.33	3.35	4.46	0.76	0.63
SBF	S2	10	120	1.45	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	1.67	1.33	3.24	4.31	0.75	0.6
SBF	S2	11	132	1.55	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	1.67	1.33	3.15	4.19	0.75	0.6
SBF	S2	12	144	1.66	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	1.67	1.33	3.07	4.08	0.75	0.6
SBF	S2	13	156	1.76	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.034	0.064	0.102	1.65	1.67	1.33	3	3.99	0.75	0.6
SBF	S2	14	168	1.86	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	1.67	1.33	3	3.99	0.75	0.6
SBF	S2	15	180	1.96	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	1.67	1.33	3	3.99	0.75	0.6
SLF	S3	1	15	0.35	0.1	0.1	0.125	0.15	0.1	0.14	0.1	0.1	0.1	0.15	0.219	0.3	2.7	2	1.5	6	7.98	0.8	0.75
SLF	S3	2	25	0.39	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2.5	2	1.5	6	7.98	0.8	0.75
SLF	S3	3	35	0.5	0.1	0.1	0.125	0.15	0.1	0.1	0.1	0.1	0.1	0.15	0.219	0.3	2.25	2	1.5	4.94	6.57	0.8	0.75

Table D.1: Building specific properties for capacity curve development

MBT		No. Storeys	H_R (ft)	T_e (s)	Basic score, C_s				Pre-code modifier, C_s				Post-benchmark modifier, C_s				Gamma, γ	Lambda, λ		Ductility, μ		Modal factors	
NRC	FEMA				VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX		Basic Score	Pre-code	Basic Score	Post-benchmark	α_1	α_2
SCW	S4	1	14	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.83	1.42	6	7.98	0.8	0.75
SCW	S4	2	24	0.35	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.83	1.42	6	7.98	0.8	0.75
SCW	S4	3	36	0.44	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.83	1.42	4.94	6.57	0.8	0.75
SCW	S4	4	48	0.55	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	1.83	1.42	4.41	5.87	0.8	0.75
SCW	S4	5	60	0.65	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	1.83	1.42	4.07	5.41	0.8	0.75
SCW	S4	6	72	0.74	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.174	1.8	1.83	1.42	3.82	5.08	0.79	0.72
SCW	S4	7	84	0.84	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	1.83	1.42	3.63	4.83	0.78	0.69
SCW	S4	8	96	0.92	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	1.83	1.42	3.48	4.63	0.77	0.66
SCW	S4	9	108	1.01	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	1.83	1.42	3.35	4.46	0.76	0.63
SCW	S4	10	120	1.09	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	1.83	1.42	3.24	4.31	0.75	0.6
SCW	S4	11	132	1.17	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.083	0.117	1.65	1.83	1.42	3.15	4.19	0.75	0.6
SCW	S4	12	144	1.25	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	1.83	1.42	3.07	4.08	0.75	0.6
SCW	S4	13	156	1.33	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.034	0.064	0.102	1.65	1.83	1.42	3	3.99	0.75	0.6
SCW	S4	14	168	1.4	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	1.83	1.42	3	3.99	0.75	0.6
SCW	S4	15	180	1.48	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	1.83	1.42	3	3.99	0.75	0.6
SIW	S5	1	14	0.35	0.055	0.055	0.055	0.055	0.055	0.055	0.055	0.055					2.7	1.67	1.33	6	7.98	0.8	0.75
SIW	S5	2	24	0.35	0.046	0.046	0.049	0.046	0.046	0.046	0.046	0.046					2.5	1.67	1.33	6	7.98	0.8	0.75
SIW	S5	3	36	0.44	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04					2.25	1.67	1.33	4.94	6.57	0.8	0.75
SIW	S5	4	48	0.55	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035					2	1.67	1.33	4.41	5.87	0.8	0.75
SIW	S5	5	60	0.65	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032					1.88	1.67	1.33	4.07	5.41	0.8	0.75
SIW	S5	6	72	0.74	0.029	0.029	0.029	0.029	0.029	0.029	0.029	0.029					1.8	1.67	1.33	3.82	5.08	0.79	0.72
SIW	S5	7	84	0.84	0.036	0.026	0.026	0.026	0.026	0.026	0.026	0.026					1.75	1.67	1.33	3.63	4.83	0.78	0.69
SIW	S5	8	96	0.92	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	NA				1.71	1.67	1.33	3.48	4.63	0.77	0.66
SIW	S5	9	108	1.01	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022					1.69	1.67	1.33	3.35	4.46	0.76	0.63
SIW	S5	10	120	1.09	0.021	0.021	0.021	0.021	0.021	0.021	0.021	0.021					1.67	1.67	1.33	3.24	4.31	0.75	0.6
SIW	S5	11	132	1.17	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019					1.65	1.67	1.33	3.15	4.19	0.75	0.6
SIW	S5	12	144	1.25	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018					1.65	1.67	1.33	3.07	4.08	0.75	0.6
SIW	S5	13	156	1.33	0.017	0.017	0.017	0.017	0.017	0.017	0.017	0.017					1.65	1.67	1.33	3	3.99	0.75	0.6
SIW	S5	14	168	1.4	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016					1.65	1.67	1.33	3	3.99	0.75	0.6
SIW	S5	15	180	1.48	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015					1.65	1.67	1.33	3	3.99	0.75	0.6
CMF	C1	1	12	0.4	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	2	1.5	6	7.98	0.75	0.75
CMF	C1	2	20	0.4	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	2	1.5	6	7.98	0.75	0.75
CMF	C1	3	30	0.48	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	2	1.5	4.94	6.57	0.75	0.75
CMF	C1	4	40	0.62	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	2	1.5	4.41	5.87	0.75	0.75
CMF	C1	5	50	0.76	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	2	1.5	4.07	5.41	0.75	0.75
CMF	C1	6	60	0.89	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	2	1.5	3.82	5.08	0.73	0.72
CMF	C1	7	70	1.03	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	2	1.5	3.63	4.83	0.71	0.69
CMF	C1	8	80	1.16	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	2	1.5	3.48	4.63	0.69	0.66
CMF	C1	9	90	1.29	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	2	1.5	3.35	4.46	0.67	0.63
CMF	C1	10	100	1.41	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	2	1.5	3.24	4.31	0.65	0.6
CMF	C1	11	110	1.54	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	2	1.5	3.15	4.19	0.65	0.6
CMF	C1	12	120	1.67	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	2	1.5	3.07	4.08	0.65	0.6
CMF	C1	13	130	1.79	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.0.34	0.064	0.102	1.65	2	1.5	3	3.99	0.65	0.6
CMF	C1	14	140	1.91	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	2	1.5	3	3.99	0.65	0.6
CMF	C1	15	150	2.04	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	2	1.5	3	3.99	0.65	0.6
CSW	C2	1	12	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	2	1.33	6	7.98	0.8	0.75

Table D.1: Building specific properties for capacity curve development

MBT		No. Storeys	H_R (ft)	T_e (s)	Basic score, C_s				Pre-code modifier, C_s				Post-benchmark modifier, C_s				Gamma, γ	Lambda, λ		Ductility, μ		Modal factors	
NRC	FEMA				VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX		Basic Score	Pre-code	Basic Score	Post-benchmark	α_1	α_2
CSW	C2	2	20	0.35	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	2	1.33	6	7.98	0.8	0.75
CSW	C2	3	30	0.39	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	2	1.33	4.94	6.57	0.8	0.75
CSW	C2	4	40	0.48	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	2	1.33	4.41	5.87	0.8	0.75
CSW	C2	5	50	0.57	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	2	1.33	4.07	5.41	0.8	0.75
CSW	C2	6	60	0.65	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	2	1.33	3.82	5.08	0.79	0.72
CSW	C2	7	70	0.73	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	2	1.33	3.63	4.83	0.78	0.69
CSW	C2	8	80	0.81	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	2	1.33	3.48	4.63	0.77	0.66
CSW	C2	9	90	0.88	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	2	1.33	3.35	4.46	0.76	0.63
CSW	C2	10	100	0.95	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	2	1.33	3.24	4.31	0.75	0.6
CSW	C2	11	110	1.02	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	2	1.33	3.15	4.19	0.75	0.6
CSW	C2	12	120	1.09	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	2	1.33	3.07	4.08	0.75	0.6
CSW	C2	13	130	1.16	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.0.34	0.064	0.102	1.65	2	1.33	3	3.99	0.75	0.6
CSW	C2	14	140	1.23	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	2	1.33	3	3.99	0.75	0.6
CSW	C2	15	150	1.29	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	2	1.33	3	3.99	0.75	0.6
CIW	C3	1	12	0.35	0.055	0.055	0.055	0.055	0.055	0.055	0.055	0.055					2.7	1.83	1.42	6	7.98	0.8	0.75
CIW	C3	2	20	0.35	0.046	0.046	0.049	0.046	0.046	0.046	0.046	0.046					2.5	1.83	1.42	6	7.98	0.8	0.75
CIW	C3	3	30	0.39	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04					2.25	1.83	1.42	4.94	6.57	0.8	0.75
CIW	C3	4	40	0.48	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035					2	1.83	1.42	4.41	5.87	0.85	0.75
CIW	C3	5	50	0.57	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032					1.88	1.83	1.42	4.07	5.41	0.8	0.75
CIW	C3	6	60	0.65	0.029	0.029	0.029	0.029	0.029	0.029	0.029	0.029					1.8	1.83	1.42	3.82	5.08	0.79	0.72
CIW	C3	7	70	0.73	0.036	0.026	0.026	0.026	0.026	0.026	0.026	0.026					1.75	1.83	1.42	3.63	4.83	0.78	0.69
CIW	C3	8	80	0.81	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024		NA			1.71	1.83	1.42	3.48	4.63	0.77	0.66
CIW	C3	9	90	0.88	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022					1.69	1.83	1.42	3.35	4.46	0.76	0.63
CIW	C3	10	100	0.95	0.021	0.021	0.021	0.021	0.021	0.021	0.021	0.021					1.67	1.83	1.42	3.24	4.31	0.75	0.6
CIW	C3	11	110	1.02	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019					1.65	1.83	1.42	3.15	4.19	0.75	0.6
CIW	C3	12	120	1.09	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018					1.65	1.83	1.42	3.07	4.08	0.75	0.6
CIW	C3	13	130	1.16	0.017	0.017	0.017	0.017	0.017	0.017	0.017	0.017					1.65	1.83	1.42	3	3.99	0.75	0.6
CIW	C3	14	140	1.23	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016					1.65	1.83	1.42	3	3.99	0.75	0.6
CIW	C3	15	150	1.29	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015					1.65	1.83	1.42	3	3.99	0.75	0.6
PCW	PC1	1	15	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.33	1.17	6	7.98	0.75	0.75
PCW	PC1	2	25	0.39	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.33	1.17	6	7.98	0.7	0.75
PCW	PC1	3	35	0.5	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.33	1.17	4.94	6.57	0.75	0.75
PCF	PC2	1	12	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.67	1.33	6	7.98	0.8	0.75
PCF	PC2	2	20	0.35	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.67	1.33	6	7.98	0.8	0.75
PCF	PC2	3	30	0.39	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.67	1.33	4.94	6.57	0.8	0.75
PCF	PC2	4	40	0.48	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	1.67	1.33	4.41	5.87	0.85	0.75
PCF	PC2	5	50	0.57	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	1.67	1.33	4.07	5.41	0.8	0.75
PCF	PC2	6	60	0.65	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	1.67	1.33	3.82	5.08	0.79	0.72
PCF	PC2	7	70	0.73	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	1.67	1.33	3.63	4.83	0.78	0.69
PCF	PC2	8	80	0.81	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	1.67	1.33	3.48	4.63	0.77	0.66
PCF	PC2	9	90	0.88	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	1.67	1.33	3.35	4.46	0.76	0.63
PCF	PC2	10	100	0.95	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	1.67	1.33	3.24	4.31	0.75	0.6
PCF	PC2	11	110	1.02	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	1.67	1.33	3.15	4.19	0.75	0.6
PCF	PC2	12	120	1.09	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	1.67	1.33	3.07	4.08	0.75	0.6
PCF	PC2	13	130	1.16	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.0.34	0.064	0.102	1.65	1.67	1.33	3	3.99	0.75	0.6
PCF	PC2	14	140	1.23	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	1.67	1.33	3	3.99	0.75	0.6

Table D.1: Building specific properties for capacity curve development

MBT		No. Storeys	H_R (ft)	T_e (s)	Basic score, C_s				Pre-code modifier, C_s				Post-benchmark modifier, C_s				Gamma, γ	Lambda, λ		Ductility, μ		Modal factors	
NRC	FEMA				VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX		Basic Score	Pre-code	Basic Score	Post-benchmark	α_1	α_2
PCF	PC2	15	150	1.29	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	1.67	1.33	3	3.99	0.75	0.6
RML	RM1	1	12	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.67	1.33	6	7.98	0.8	0.75
RML	RM1	2	20	0.35	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.67	1.33	6	7.98	0.8	0.75
RML	RM1	3	30	0.39	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.67	1.33	4.94	6.57	0.8	0.75
RML	RM1	4	40	0.48	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	1.67	1.33	4.41	5.87	0.85	0.75
RML	RM1	5	50	0.57	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	1.67	1.33	4.07	5.41	0.8	0.75
RML	RM1	6	60	0.65	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	1.67	1.33	3.82	5.08	0.79	0.72
RML	RM1	7	70	0.73	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	1.67	1.33	3.63	4.83	0.78	0.69
RML	RM1	8	80	0.81	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	1.67	1.33	3.48	4.63	0.77	0.66
RML	RM1	9	90	0.88	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	1.67	1.33	3.35	4.46	0.76	0.63
RML	RM1	10	100	0.95	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	1.67	1.33	3.24	4.31	0.75	0.6
RML	RM1	11	110	1.02	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	1.67	1.33	3.15	4.19	0.75	0.6
RML	RM1	12	120	1.09	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	1.67	1.33	3.07	4.08	0.75	0.6
RML	RM1	13	130	1.16	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.0.34	0.064	0.102	1.65	1.67	1.33	3	3.99	0.75	0.6
RML	RM1	14	140	1.23	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	1.67	1.33	3	3.99	0.75	0.6
RML	RM1	15	150	1.29	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	1.67	1.33	3	3.99	0.75	0.6
RMC	RM2	1	12	0.35	0.055	0.055	0.082	0.109	0.055	0.055	0.055	0.055	0.055	0.109	0.204	0.327	2.7	1.67	1.33	6	7.98	0.8	0.75
RMC	RM2	2	20	0.35	0.046	0.046	0.069	0.092	0.046	0.046	0.046	0.046	0.046	0.092	0.173	0.276	2.5	1.67	1.33	6	7.98	0.8	0.75
RMC	RM2	3	30	0.39	0.04	0.04	0.06	0.08	0.04	0.04	0.04	0.04	0.04	0.08	0.15	0.24	2.25	1.67	1.33	4.94	6.57	0.8	0.75
RMC	RM2	4	40	0.48	0.036	0.036	0.053	0.071	0.035	0.035	0.035	0.035	0.036	0.071	0.133	0.213	2	1.67	1.33	4.41	5.87	0.85	0.75
RMC	RM2	5	50	0.57	0.032	0.032	0.047	0.063	0.032	0.032	0.032	0.032	0.032	0.063	0.118	0.189	1.88	1.67	1.33	4.07	5.41	0.8	0.75
RMC	RM2	6	60	0.65	0.029	0.029	0.043	0.057	0.029	0.029	0.029	0.029	0.029	0.057	0.107	0.171	1.8	1.67	1.33	3.82	5.08	0.79	0.72
RMC	RM2	7	70	0.73	0.026	0.026	0.039	0.052	0.026	0.026	0.026	0.026	0.026	0.052	0.098	0.156	1.75	1.67	1.33	3.63	4.83	0.78	0.69
RMC	RM2	8	80	0.81	0.024	0.024	0.036	0.048	0.024	0.024	0.024	0.024	0.024	0.048	0.09	0.144	1.71	1.67	1.33	3.48	4.63	0.77	0.66
RMC	RM2	9	90	0.88	0.022	0.022	0.033	0.044	0.022	0.022	0.022	0.022	0.022	0.044	0.083	0.132	1.69	1.67	1.33	3.35	4.46	0.76	0.63
RMC	RM2	10	100	0.95	0.021	0.021	0.031	0.041	0.021	0.021	0.021	0.021	0.021	0.041	0.077	0.123	1.67	1.67	1.33	3.24	4.31	0.75	0.6
RMC	RM2	11	110	1.02	0.02	0.02	0.029	0.039	0.019	0.019	0.019	0.019	0.02	0.039	0.073	0.117	1.65	1.67	1.33	3.15	4.19	0.75	0.6
RMC	RM2	12	120	1.09	0.018	0.018	0.027	0.036	0.018	0.018	0.018	0.018	0.018	0.036	0.068	0.108	1.65	1.67	1.33	3.07	4.08	0.75	0.6
RMC	RM2	13	130	1.16	0.017	0.017	0.026	0.034	0.017	0.017	0.017	0.017	0.017	0.0.34	0.064	0.102	1.65	1.67	1.33	3	3.99	0.75	0.6
RMC	RM2	14	140	1.23	0.016	0.016	0.024	0.032	0.016	0.016	0.016	0.016	0.016	0.032	0.06	0.096	1.65	1.67	1.33	3	3.99	0.75	0.6
RMC	RM2	15	150	1.29	0.016	0.016	0.023	0.031	0.016	0.016	0.016	0.016	0.016	0.031	0.058	0.093	1.65	1.67	1.33	3	3.99	0.75	0.6
URM	URM	1	12	0.35	0.055	0.055	0.055	0.055	0.055	0.055	0.055	0.055					2.7	1.33	1.17	6	7.98	0.75	0.75
URM	URM	2	20	0.35	0.046	0.046	0.049	0.046	0.046	0.046	0.046	0.046					2.5	1.33	1.17	6	7.98	0.75	0.75
URM	URM	3	30	0.39	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04					2.25	1.33	1.17	4.94	6.57	0.75	0.75
URM	URM	4	40	0.48	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035					2	1.33	1.17	4.41	5.87	0.75	0.75
URM	URM	5	50	0.57	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032					1.88	1.33	1.17	4.07	5.41	0.75	0.75
URM	URM	6	60	0.65	0.029	0.029	0.029	0.029	0.029	0.029	0.029	0.029					1.8	1.33	1.17	3.82	5.08	0.73	0.72
URM	URM	7	70	0.73	0.036	0.026	0.026	0.026	0.026	0.026	0.026	0.026					1.75	1.33	1.17	3.63	4.83	0.71	0.69
URM	URM	8	80	0.81	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	NA				1.71	1.33	1.17	3.48	4.63	0.69	0.66
URM	URM	9	90	0.88	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022					1.69	1.33	1.17	3.35	4.46	0.67	0.63
URM	URM	10	100	0.95	0.021	0.021	0.021	0.021	0.021	0.021	0.021	0.021					1.67	1.33	1.17	3.24	4.31	0.65	0.6
URM	URM	11	110	1.02	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019					1.65	1.33	1.17	3.15	4.19	0.65	0.6
URM	URM	12	120	1.09	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018					1.65	1.33	1.17	3.07	4.08	0.65	0.6
URM	URM	13	130	1.16	0.017	0.017	0.017	0.017	0.017	0.017	0.017	0.017					1.65	1.33	1.17	3	3.99	0.65	0.6
URM	URM	14	140	1.23	0.016	0.016	0.016	0.016	0.016	0.016	0.016	0.016					1.65	1.33	1.17	3	3.99	0.65	0.6
URM	URM	15	150	1.29	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015					1.65	1.33	1.17	3	3.99	0.65	0.6

Table D.1: Building specific properties for capacity curve development

MBT		No. Storeys	H_R (ft)	T_e (s)	Basic score, C_s				Pre-code modifier, C_s				Post-benchmark modifier, C_s				Gamma, γ	Lambda, λ		Ductility, μ		Modal factors	
NRC	FEMA				VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX	VL,L	M	MH	H,VH, VHX		Basic Score	Pre-code	Basic Score	Post-benchmark	α_1	α_2
MH	MH	1	10	0.35	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.125	0.15	1.5	2	1.5	6	7.98	1	1

† In FEMA P-154, W1 and W2 *building types* can be no more than five *storeys* high, however, in the NBC, the maximum is six. Extrapolation is performed based on natural periods of W1 and W2 *building type*s, which have five or less *storeys*.

Table D.2: Elastic damping (β_E)	
Building type	**Elastic damping (%)**
WLF	10
WPB	10
SMF	5
SBF	5
SLF	5
SCW	5
SIW	7
CMF	7
CSW	7
CIW	7
PCW	7
PCF	7
RML	7
RMC	7
MH	5

Table D.3: Degradation factor (κ_d)				
Type	**VL,L**	**M**	**MH**	**H,VH,VHX**
Basic score	0.3	0.3	0.4	0.4
Pre-code	0.3	0.3	0.4	0.4
Post-benchmark	0.45	0.45	0.6	0.668

Table D.4: Interstorey drift ratios (Δ_c)

	WLF, WPB				SMF, SBF, CSW				SLF, SCW, PCF, RML, RMC				CMF				PCW				SIW, CIW, URM				MH			
Property	**VL,L**	**M**	**MH**	**H,VH, VHX**	**LVL**	**M**	**MH**	**H,VH, VHX**	**VL,L**	**M**	**MH**	**H,VH, VHX**	**VL,L**	**M**	**MH**	**H,VH, VHX**	**VL,L**	**M**	**MH**	**H,VH, VHX**	**VL,L**	**M**	**MH**	**H,VH, VHX**	**VL,L**	**M**	**MH**	**H,VH, VHX**
Basic score	0.075	0.075	0.075	0.075	0.05	0.05	0.055	0.06	0.044	0.044	0.049	0.053	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.056	0.07	0.07	0.07
Plan irregularity	0.038	0.038	0.038	0.038	0.025	0.025	0.028	0.03	0.022	0.022	0.025	0.027	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.028	0.035	0.035	0.035
Severe or moderate vertical irregularity	0.06	0.06	0.06	0.06	0.04	0.04	0.045	0.05	0.035	0.035	0.04	0.044	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.028	0.056	0.07	0.07	0.07
Pre-code modifiers	0.075	0.075	0.075	0.075	0.05	0.05	0.05	0.05	0.044	0.044	0.044	0.044	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.056	0.056	0.063	0.07
Post-benchmark modifiers	0.094	0.094	0.113	0.125	0.063	0.063	0.083	0.1	0.055	0.055	0.073	0.089	0.063	0.063	0.083	0.1	0.055	0.055	0.073	0.089	NA				0.07	0.088	0.105	0.117

Table D.5: Modal factor (α3)			
No. Storeys	**Basic Score**	**Moderate vertical irregularity**	**Severe vertical irregularity**
1	1	1	1
2	1.21	1.62	2.03
3	1.35	2.04	2.73
4	1.45	2.36	3.27
5	1.65	2.63	4
6	1.62	2.87	4
7	1.69	3.07	4
8	1.75	3.26	4
9	1.81	3.43	4
10	1.86	3.59	4
11	1.91	3.73	4
12	1.96	3.87	4
13	2	4	4
14	2.04	4	4
15	2.08	4	4

Table D.6: Logarithmic standard deviations ($\beta_{C,D}$)					
No. Storeys	**All building types Basic scores**	**All building types Pre-code**	**SMF Post-benchmark**	**All other building types (except SMF) Post-benchmark**	**All building types Post-disaster**
1	0.95	1	0.75	0.85	0.90
2	0.95	1	0.75	0.85	0.90
3	0.95	1	0.75	0.85	0.90
4	0.94	0.99	0.75	0.84	0.89
5	0.93	0.98	0.75	0.83	0.88
6	0.92	0.97	0.75	0.82	0.87
7	0.91	0.96	0.75	0.81	0.86
8	0.9	0.95	0.75	0.8	0.85
9	0.89	0.94	0.75	0.79	0.84
10	0.88	0.93	0.75	0.78	0.83
11	0.87	0.92	0.75	0.77	0.82
12	0.86	0.91	0.75	0.76	0.81
13	0.85	0.9	0.75	0.75	0.8
14	0.85	0.9	0.75	0.75	0.8
15	0.85	0.9	0.75	0.75	0.8

Table D.7: Collapse factor (CF)				
Property	**WLF-WPB-MH**	**SMF-SBF-SLF-SCW-SIW**	**CMF-CSW-CIW-RML-RMC**	**PCW-PCF-URM**
Basic Score	0.05	0.08	0.13	0.15
Severe vertical irregularity	0.2	0.3	0.5	0.6
Moderate vertical irregularity and plan irregularity	0.1	0.15	0.25	0.3

Table D.8: Building capacity and fragility parameters for different importance categories			
Construction importance	**Capacity curve**	**Fragility curve**	
	$\boldsymbol{I_E \times C_S}$	$\boldsymbol{k_\Delta \times S_{d,C}}$	$\boldsymbol{\beta_{CD}}$
Low importance ($I_E = 0.8$)	0.8 C_S	0.9 $S_{d,C}$	Pre-code
Normal importance ($I_E = 1.0$)	1.0 C_S	1.0 $S_{d,C}$	Basic score
Post-disaster ($I_E = 1.5$)	1.5 C_S	1.25 $S_{d,C}$	Post-disaster

Table D.9: Remaining occupancy time factor (κ)	
Remaining occupancy time	κ
30	0.83
25	0.76
20	0.67
15	0.57
10	0.44
5	0.28

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APPENDIX E CALCULATION OF AREA WITHIN ELLIPTICAL REGION OF BUILDING CAPACITY CURVE

The *building* capacity curve between the yield capacity point and the ultimate capacity point is assumed to have an elliptical shape that is tangent to the elastic segment at yield capacity point and tangent to the plastic segment at the ultimate capacity point, which is determined by:

$$\frac{(D-D_u)^2}{a^2}+\frac{(A-k)^2}{b^2}=1 \tag{E.1}$$

where the values of a, b, and k are calculated by:

$$a=\sqrt{\frac{D_y}{A_y}\frac{\left(D_u-D_y\right)}{\left(A_y-k\right)}b^2}$$

$$b=A_u-k$$

$$k=\frac{A_u^2-A_y^2+\frac{A_y^2}{D_y}\left(D_y-D_u\right)}{2\left(A_u-A_y\right)+\frac{A_y}{D_y}\left(D_y-D_u\right)}$$

where A_y and A_u are elastic and ultimate spectral accelerations, and D_y and D_u are elastic and ultimate spectral displacements. A_y, A_u, D_y and D_u values are determined according to this appendix. A and D are the spectral acceleration and spectral displacement within the elliptical region of the capacity curve.

Spectral acceleration A between A_y and A_u is obtained as follows:

$$A=b\sqrt{1-\frac{(D-D_u)^2}{a^2}}+k \tag{E.2}$$

Therefore, the area under the curve between D_y and S_d can be evaluated by:

$$\begin{aligned} Area_{ellip} &= \int_{D_y}^{S_d} A\,\mathrm{d}D=\int_{D_y}^{S_d}\left(b\sqrt{1-\frac{(D-D_u)^2}{a^2}}+k\right)\mathrm{d}D \\ &= b\int_{D_y}^{S_d}\sqrt{1-\frac{(D-D_u)^2}{a^2}}\,\mathrm{d}D+k\times\left(S_d-D_y\right) \end{aligned} \tag{E.3}$$

Let

$$\sin t = \frac{D - D_u}{a}$$

Hence,

$$D = a \times \sin t + D_u$$

Differentiating D gives:

$$\mathrm{d}D = a \times \cos t$$

Substituting $\sin t$ and $\mathrm{d}D$ into Eq. (B.3) results in:

$$\begin{aligned} Area_{ellip} &= k \times (S_d - D_y) + ab\int_{t_1}^{t_2} \sqrt{1 - \sin^2 t} \times \cos t \, \mathrm{d}t \\ &= k \times (S_d - D_y) + ab\int_{t_1}^{t_2} \cos^2 t \, \mathrm{d}t \\ &= k \times (S_d - D_y) + ab\int_{t_1}^{t_2} \frac{1 + \cos 2t}{2} \mathrm{d}t \\ &= k \times (S_d - D_y) + \frac{ab}{2}(t_2 - t_1) + \frac{ab}{4}(\sin 2t_2 - \sin 2t_1) \end{aligned} \tag{E.4}$$

where:

$$t_1 = \sin^{-1} \frac{D_y - D_u}{a}$$

$$t_2 = \sin^{-1} \frac{S_d - D_u}{a}$$

Therefore, incremental area A_{inc} is calculated by:

$$\begin{aligned} A_{inc} &= S_a \times (S_d - D_y) - Area_{ellip} \\ &= (S_a - k) \times (S_d - D_y) - \frac{ab}{2}(t_2 - t_1) - \frac{ab}{4}(\sin 2t_2 - \sin 2t_1) \end{aligned}$$

When *building* capacity approaches ultimate capacity point, i.e., $S_d = D_u$,

$$t_2 = \sin^{-1} \frac{D_u - D_u}{a} = 0$$

$$A_{inc} = (S_a - k) \times (D_u - D_y) + \frac{ab}{2} \times t_1 + \frac{ab}{4} \times \sin 2t_1$$