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Introduction

The Commentaries presented herein serve to provide designers with detailed design information that will assist them in the application of Part 4 of Division B of the National Building Code of Canada (NBC) 2015. They contain background information and suggested approaches to certain design issues; their provisions are not mandatory requirements of the NBC.

The information set forth in these Commentaries does not cover all conditions and types of structures that occur in practice. For unusual types of structures, specialized information, such as theoretical studies, model tests or wind tunnel experiments, may be required to provide adequate design values. It should also be noted that new information can become available at any time. As such, designers are encouraged to obtain the latest and most appropriate design information available.

The Commentaries were updated by the 2010–2015 and 2015–2020 Standing Committees on Structural Design and Earthquake Design of the Canadian Commission on Building and Fire Codes to reflect technical changes made to Part 4 of the NBC 2015.

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Referenced Standards

The following table lists the North American and ISO standards that are referenced in this Guide along with the applicable editions and the location of each reference. Many of these standards are also referenced in one of the National Model Codes, most notably, in the National Building Code. Other types of documents (articles, reports, etc.) referenced in this Guide are listed in the "References" section at the end of each Commentary.

Standards Referenced in the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B)

Standard Number(1)	Standard Title	Guide Reference	
AAMA 501.6-09	Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System	Commentary J	Para. 248
ACI 355.2-07	Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary	Commentary J	Para. 237
ACI 355.4-11	Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary	Commentary J	Para. 237
ACI 543R-74	Recommendations for Design, Manufacture, and Installation of Concrete Piles	Commentary K	Para. 115 Table K-12
ANSI MH16.1-2012	Design, Testing and Utilization of Industrial Steel Storage Racks	Commentary J	Para. 243
API 620-2013	Design and Construction of Large, Welded, Low-Pressure Storage Tanks	Commentary J	Para. 247
API 650-2013	Welded Tanks for Oil Storage	Commentary J	Para. 247
ASCE/SEI 7-10	Minimum Design Loads for Buildings and Other Structures	Commentary G	Para. 6
		Commentary I	Para. 68 Table I-2
		Commentary J	Para. 79 Para. 153 Para. 248 Para. 255 Para. 275
ASCE/SEI 11-99	Guideline for Structural Condition Assessment of Existing Buildings	Commentary L	Para. 55
ASCE/SEI 41-13	Seismic Evaluation and Retrofit of Existing Buildings	Commentary J	Para. 78 Para. 179 Para. 217 Para. 255 Para. 275
		Commentary L	Para. 39 Para. 44 Para. 45 Para. 46
ASCE/SEI 49-12	Wind Tunnel Testing for Buildings and Other Structures	Commentary G	Para. 25 Para. 45
		Commentary I	Para. 4 Para. 70
ASME A17.1-2010/CSA B44-10	Safety Code for Elevators and Escalators	Commentary J	Para. 246
ASTM D 1143/D 1143M-07	Deep Foundations Under Static Axial Compressive Load	Commentary K	Table K-11

Standards Referenced in the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B) (Continued)

Standard Number[1]	Standard Title	Guide Reference	
ASTM E 985-06	Permanent Metal Railing Systems and Rails for Buildings	Commentary F	Para. 23
ASTM E 1300-12ae1	Determining Load Resistance of Glass in Buildings	Commentary I	Para. 68 Table I-2 Table I-3
ANSI/AWWA D100-96	Welded Steel Tanks for Water Storage	Commentary J	Para. 247
ANSI/AWWA D110-95	Wire- and Strand-Wound Circular, Prestressed Concrete Water Tanks	Commentary J	Para. 247
ANSI/AWWA D115-95	Circular Prestressed Concrete Tanks with Circumferential Tendons	Commentary J	Para. 247
CAN/CGSB-12.20-M89	Structural Design of Glass for Buildings	Commentary I	Para. 68 Table I-2 Table I-3
CSA A23.1-14	Concrete Materials and Methods of Concrete Construction	Commentary K	Para. 118
		Commentary L	Para. 54
CSA A23.3-14	Design of Concrete Structures	Commentary A	Table A-4
		Commentary D	Para. 2
		Commentary E	Table E-1
		Commentary J	Para. 92 Para. 118 Para. 132 Para. 204 Para. 205 Para. 207 Para. 217 Para. 218 Para. 220 Para. 237 Para. 256 Para. 277
		Commentary K	Para. 78 Para. 120 Table K-14
		Commentary L	Para. 46 Para. 59 Para. 67
CSA A344-17	User Guide for Steel Storage Racks	Commentary J	Para. 243
CAN/CSA-O80 Series-08	Wood Preservation	Commentary K	Table K-12
CSA O86-14	Engineering Design in Wood	Commentary A	Table A-4
		Commentary D	Table D-1
		Commentary J	Para. 88 Para. 118 Para. 204 Para. 206
CSA S6-14	Canadian Highway Bridge Design Code	Commentary A	Para. 22
		Commentary E	Para. 4
		Commentary G	Para. 55
		Commentary J	Para. 255
		Commentary K	Para. 17 Para. 26 Para. 53 Para. 54 Para. 78 Para. 82
		Commentary L	Para. 60

Standards Referenced in the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B) (Continued)

Standard Number[1]	Standard Title	Guide Reference	
CSA S6.1-14	Commentary on CSA S6-14, Canadian Highway Bridge Design Code	Commentary D	Para. 7
		Commentary K	Para. 17 Para. 54 Para. 82
CSA S16-14	Design of Steel Structures	Commentary A	Para. 24 Para. 26 Table A-4
		Commentary D	Table D-1
		Commentary J	Para. 118 Para. 204 Para. 206 Para. 207 Para. 243
		Commentary L	Para. 13
CSA S37-13	Antennas, Towers, and Antenna-Supporting Structures	Commentary G	Para. 55
		Commentary I	Para. 67
CSA S136-12	North American Specification for the Design of Cold-Formed Steel Structural Members	Commentary E	Table E-1
		Commentary J	Para. 151
CAN/CSA-S157-05/S157.1-05	Strength Design in Aluminum/Commentary on CSA S157-05, Strength Design in Aluminum	Commentary A	Para. 26
CSA S304-14	Design of Masonry Structures	Commentary A	Table A-4
		Commentary D	Table D-1
		Commentary E	Table E-1
		Commentary I	Para. 82
		Commentary J	Para. 204 Para. 256 Para. 277
CSA S408-81	Guidelines for the Development of Limit States Design	Commentary A	Para. 10
CSA S413-14	Parking Structures	Commentary F	Para. 27 Para. 29
		Commentary L	Para. 54
CSA S448.1-10	Repair of Reinforced Concrete in Buildings and Parking Structures	Commentary L	Para. 54
CSA S478-95	Guideline on Durability in Buildings	Commentary L	Para. 54
CAN/CSA-S832-06	Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings	Commentary J	Para. 230 Para. 242
		Commentary L	Para. 48
CSA S850-12	Design and Assessment of Buildings Subjected to Blast Loads	Commentary A	Para. 12
CSA SPE-900-13	Solar Photovoltaic Rooftop-Installation Best Practices Guideline	Commentary I	Fig. I-8 Fig. I-9
CSA W59-03	Welded Steel Construction (Metal Arc Welding)	Commentary K	Para. 120
CSA Z240.10.1-08	Site Preparation, Foundation, and Anchorage of Manufactured Homes	Commentary I	Para. 82
FEMA 222A-1994	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions	Commentary J	Fig. J-9 Para. 71
FEMA 302-1997	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions	Commentary J	Para. 129 Para. 225
FEMA 356-2000	Prestandard and Commentary for the Seismic Rehabilitation of Buildings	Commentary J	Para. 160

Standards Referenced in the Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B) (Continued)

Standard Number(1)	Standard Title	Guide Reference	
FEMA 368-2001	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions	Commentary J	Para. 179 Para. 225 Para. 226 Para. 230 Para. 231
FEMA 369-2001	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 2: Commentary	Commentary J	Para. 127 Para. 223
FEMA 450-1-2003	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions	Commentary J	Para. 247 Para. 248
FEMA 450-2-2003	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 2: Commentary	Commentary J	Para. 248
FEMA 460-2005	Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public	Commentary J	Para. 243
FEMA P-695-2009	Quantification of Building Seismic Performance Factors	Commentary J	Para. 92
FEMA P-750-2009	NEHRP Recommended Seismic Provisions for New Buildings and Other Structures	Commentary J	Para. 255 Para. 275
FEMA P-751-2012	2009 NEHRP Recommended Seismic Provisions: Design Examples	Commentary J	Para. 255 Para. 275
ISO 2394:1986	General Principles on Reliability for Structures	Commentary A	Para. 1 Para. 10
ISO 10137:2007	Bases for Design of Structures – Serviceability of Buildings and Walkways Against Vibration	Commentary D	Para. 6
		Commentary I	Para. 77
ISO 12494:2001	Atmospheric Icing of Structures	Commentary I	Para. 67

(1) The abbreviations used in the standard numbers refer to the following standards development organizations:
- AAMA: American Architectural Manufacturers Association
- ACI: American Concrete Association
- ANSI: American National Standards Institute
- API: American Petroleum Institute
- ASCE/SEI: American Society of Civil Engineers/Structural Engineering Institute
- ASME: American Society of Mechanical Engineers
- ASTM: American Society for Testing and Materials International
- AWWA: American Water Works Association
- CGSB: Canadian General Standards Board
- CSA: CSA Group
- FEMA: Federal Emergency Management Agency
- ISO: International Organization for Standardization

Commentary A

Limit States Design

Commentary A

Limit States Design

Notable Change in this Commentary

- Update to rare loads to include blast loads (addition of reference to CSA S850) and impact loads

Limit States

1. All building structures have the same basic functional requirements, namely that they should have an acceptable level of safety against collapse both during construction and throughout the life of the building, and be serviceable during the useful life of the building. The onset of various types of collapse and unserviceability are called limit states:
   - the limit states concerning safety are called ultimate limit states (ULS) and include the exceeding of load-carrying capacity, fracture, overturning, sliding and large deformation;
   - the limit state concerning failure resulting from many load repetitions is called the fatigue limit state (FLS) and applies mostly to crane-supporting structures; and
   - the limit states concerning serviceability are called serviceability limit states (SLS) and include deflection of the structure causing building damage, deflection or local damage of the structure causing malfunction of the building, and vibration of the structure causing annoyance to the occupants or malfunction of sensitive equipment.

   Previous design methods—working stress design, plastic design, ultimate strength design—emphasized only one limit state, usually associated with a limiting stress or member strength. Limit states design recognizes all categories of failure and, more importantly, provides a unified methodology for design calculations. It takes into account, by means of separate factors, the variability of both the loads and the resistances to provide consistent probability against failure, including the consequences of failure as related to the use of the component or the structure as a whole. As well, due to the advent of lighter composite-acting construction with less stiffening and less damping from curtain walls and partitions, serviceability requirements such as deflection and vibration of the structure have become more critical in structural design, and deserve the same consideration as strength requirements. The unified methodology for design calculations is the main reason why the limit states method has been adopted internationally in ISO 2394, "General Principles on Reliability for Structures."

Methods of Analysis

2. Previous design methods put the main emphasis on a particular structural theory such as elastic or plastic theory. No particular theory, however, applies universally to all limit states and all types of construction. Elastic theory is generally applicable for serviceability limit states, the fatigue limit state, and ultimate limit states of linearly elastic systems; plastic theory is generally applicable for ultimate limit states of ductile systems; and stability analysis is generally applicable for overturning. Traditional static analysis of the structure is used mostly for the design of structures, however dynamic analysis of structures is becoming more widely used for calculating the effects of dynamic loads such as earthquake, wind and those due to human activities (see Commentary D). The appropriate theory is either implicit or indicated in the structural material standard referenced in NBC Section 4.3., or chosen by the engineer. Many standards explicitly recognize inelastic behaviour of the material, when appropriate, and also require second-order geometric effects to be taken into consideration. To calculate earthquake effects, dynamic analysis is required by NBC Subsection 4.1.8.

for all buildings except those in areas of low seismicity and those meeting certain configuration and design restrictions in areas of higher seismicity. NBC Sentences 4.1.3.2.(11) and (12) and NBC Subsection 4.1.8. all require that, for the structural analysis, consideration be given to overall stability and the displaced configuration of the structure (P-$\triangle$ effects).

3. The aim of design calculations using the limit states method is to prevent failure, that is to say, the attainment of a limit state. However, unpredictable factors such as loads and workmanship enter into the calculations, so the aim is in fact that the probability of failure be sufficiently small. The more serious the consequences of failure, the smaller its probability of occurrence should be. Satisfactory failure probabilities are achieved through the use of reliable materials, competent structural engineering, manufacture and erection, and by the use of safety and serviceability criteria in the design calculations. The safety and serviceability criteria should provide adequate human safety and serviceability on the one hand, and economy on the other hand, i.e. optimum cost-effectiveness or smaller failure probabilities.[1] This is achieved in limit states design through the statistical definition of specified loads and material properties and the use of load factors, resistance factors and importance factors.

Safety and Serviceability Criteria

4. The general form of safety criteria for the ultimate limit states used or referenced by NBC Part 4 can be expressed as follows:

$$\text{factored resistance} \geq \text{effect of factored loads, or}$$
$$\phi \mathrm{R} \geq \Sigma \alpha_{\mathrm{i}} \mathrm{S}_{\mathrm{i}} \tag{1}$$

Information on the factored loads, load combinations, and the effect of factored loads can be found in Paragraphs 9 to 26. The factored loads are selected to achieve a small probability of exceedance. The factored resistance is the calculated resistance of a member, connection or structure multiplied by a resistance factor, which takes into account the variability of material properties and dimensions, workmanship, type of failure (e.g., gradual versus sudden), and modelling uncertainty in the prediction of resistance. The factored resistances, including the resistance factor, are specified in the material design standards referenced in NBC Section 4.3.

5. The general form of criteria for the serviceability limit states can be expressed as follows:

$$\text{serviceability limit} \geq \text{effect of service loads}$$

Information on the service loads and serviceability load combinations can be found in Paragraphs 27 and 28. The serviceability limits are specified or recommended in NBC Sentences 4.1.3.5.(3) and 4.1.8.13.(3), in Commentaries D, E, I, J and K, and in the material design standards referenced in NBC Section 4.3.

Specified Loads and Resistances

6. In the limit states method, specified loads and specified material properties used to calculate resistance are defined on the basis of probability of occurrence. Values so defined are called characteristic values. Specified material properties are lower exclusion limit estimates determined based on testing representative samples of various structural materials under reference conditions applicable to their in-service behaviours. Climatic loads are based on measurements taken at weather stations and the characteristic value corresponds to the probability of exceedance per year (or its reciprocal, the return period). Characteristic values for material properties and loads used in the NBC are given in Table A-1. Where statistical information is lacking, for example for live load due to use and occupancy, the specified values correspond to the existing nominal values. For specified snow and wind loads, the annual probability of exceedance for the basic climatic data was reduced from 3.3% (30-year return period) to 2.0% (50-year return period) in the 2005 NBC to be consistent with most other countries, including the United States. The material resistance of new materials or new control methods should be defined on the basis of a 5% exclusion limit and their material stiffness should be defined on the basis of a 50% exclusion limit.

Table A-1
Characteristic Values for Loads and Material Properties in the NBC

Materials	Lower Exclusion Limit(1)
Concrete (cylinder test)	≈9%
Wood	5%
Steel (yield in tension)	1%
Masonry (for prism tests)	≈9%
Loads	Return Period
Dead	Not defined
Use and Occupancy	Not defined
Snow	50 years
Wind	50 years
Earthquake	2 500 years(2)

(1) Probability of test values being less than the nominal value.
(2) See Commentary J.

Importance Factor

7. In pre-2005 editions of the NBC, buildings were simply categorized as post-disaster buildings, regular buildings, schools, or low human occupancy buildings; building type and importance with respect to wind, snow and earthquake loads were not consistently addressed. With a view to the standardization of design requirements, Importance Categories for buildings were introduced in Sentence 4.1.2.1.(3) and Table 4.1.2.1. of the NBC 2005.

These Importance Categories cover normal buildings, buildings presenting a low hazard to human life in the event of failure, high-importance buildings and post-disaster buildings all of whose importance to the community is based on:

- the presence of hazardous materials within the buildings,
- the buildings' potential to serve as emergency shelters,
- the presence of facilities, such as emergency response facilities, that are needed immediately after an emergency or disaster, and
- the presence of public utilities, such as power or water, that are needed to assist with post-disaster recovery and whose loss of function could cause additional widespread disruption or economic loss in the community.

The Low, Normal, High, and Post-disaster Importance Categories described in NBC Table 4.1.2.1. are used in conjunction with the importance factors defined in NBC Subsections 4.1.6., Loads Due to Snow and Rain, 4.1.7., Wind Load, and 4.1.8., Earthquake Load and Effects. The ultimate limit state factors are less than 1.0 for low-importance buildings and greater than 1.0 for high-importance or post-disaster buildings, values which reflect the goal of enhanced performance for these buildings, which is critical to the community in an emergency or disaster, or the fact that they contain hazardous materials or products.

Buildings designed as post-disaster facilities should remain operational immediately after an emergency or disaster. However, the mere application of an importance factor greater than 1.0 does not necessarily ensure the operational readiness of a facility following an emergency or disaster; this can only be determined by carrying out a detailed study of what equipment and services need to be operational immediately after an emergency or disaster and of the anticipated behaviour of equipment and structural components. Such a study should address issues like what equipment should be connected to emergency power, how long emergency generators need to be able to run, how secure the fuel supply is, whether or not a stored supply of potable water is required, etc.

With respect to utilities such as water, power and sewage treatment, the "post-disaster" designation is only intended to apply to public ones (privately or publicly owned) that provide services to a community. Examples of utilities that need not be included in this designation are power-generating facilities in an industrial plant that are not connected to the public power grid, a septic tank field for a single building and a homeowner's private water purification system. Where some of the power from an industrial plant is sold to a public utility, the plant's Importance Category is determined by the utility.

In pre-2005 editions of the NBC, there was a general importance factor designated γ in NBC Subsection 4.1.3. Further to this, the treatment of importance for the structure in relation to the use and occupancy of the building was different for different loads: there was a load-specific importance factor designated I in the Subsection on earthquake loads, while the Subsection on wind loads handled importance by specifying a small annual probability of exceedance (1/100 for post-disaster buildings, 1/30 for other buildings and 1/10 for cladding); there was no load-specific importance treatment for snow loads. In order to standardize the calculation of the different loads, importance factors were established for each load specified in NBC Subsections 4.1.6. (snow and rain), 4.1.7. (wind), and 4.1.8. (earthquake) of the NBC 2005. There is no importance factor for dead loads nor is there one for live loads due to use and occupancy because the loads specified in NBC Table 4.1.5.3. already take into account the more serious consequences of failure according to type of occupancy (e.g., assembly occupancies). For buildings in the Low Importance Category, however, a factor of 0.8 may be applied to the specified live load due to use and occupancy, as stated in NBC Sentences 4.1.5.1.(2) and 4.1.5.2.(2). The importance factors are summarized in Table A-2.

8. The importance factor for the serviceability limit states is taken equal to or less than 1.0 because of the less serious consequences of failure and because design criteria for serviceability are more subjective than for strength and stability.

Table A-2
Importance Factors

Importance Category	Earthquake, I_E		Wind, I_W		Snow, I_S	
	ULS	SLS	ULS	SLS	ULS	SLS
Low(1)	0.8	(2)	0.8	0.75	0.8	0.9
Normal	1.0		1.0	0.75	1.0	0.9
High	1.3		1.15	0.75	1.15	0.9
Post-disaster	1.5		1.25	0.75	1.25	0.9

(1) A factor of 0.8 may be applied to live load due to use and occupancy for buildings in the Low Importance Category.
(2) See Commentary J.

Load Combinations

9. Limit states criteria specified in NBC Article 4.1.3.2. and recommended in this Commentary are intended to provide an acceptable and relatively uniform degree of reliability in the design of structural members under different load combinations. The criteria take into consideration the probability of failure due to the simultaneous occurrence of the loads specified in NBC Subsections 4.1.4. to 4.1.8. Paragraphs 10 to 18 explain and provide guidance on the load combinations given in NBC Tables 4.1.3.2.-A and -B. Paragraphs 19 to 25 provide guidance for situations where the load combinations given in Tables 4.1.3.2.-A and -B do not apply. For the structural evaluation of building structures not within the scope of the standards listed in NBC Section 4.3., including building envelopes, the generalized load combinations stated in Paragraph 10 are recommended. Paragraphs 10 and 26 to 28 provide guidance for the determination of loads and load combinations for the fatigue and serviceability limit states.

Generalized Load Combinations

10. Structural loads can be divided into three categories: permanent loads (such as dead load and earth pressure), variable loads (such as use and occupancy, snow and wind loads), and rare loads or situations (such as earthquake or fire). In general, load combinations can be determined by splitting the loads specified in NBC Part 4 into two components (see CSA S408, "Guidelines for the Development of Limit States Design," and Reference [2]): a sustained or frequently occurring component (e.g., dead load, earth pressure, sustained live load) and a transient component, which acts rarely and for a short time only (e.g., impact, wind, earthquake, short-term accumulation of people and/or objects). Because the transient components of different loads are unlikely to occur simultaneously, the critical load combination for a given structural effect is estimated by combining the factored permanent loads with the factored variable or rare load having the largest transient component, plus the sustained or frequent components of all other variable loads. This principle, called the companion action principle[2] and recommended by ISO 2394 has been applied to determine the following generalized factored load combinations for both the ultimate and serviceability limit states.

Load Combinations for Variable Loads

11. Where all loads are permanent or variable, the load combinations are:

$$\Sigma\alpha_{G_i}G_i + \alpha_{Q_l}Q_l + \Sigma\alpha_{CQ_i}Q_i \tag{2}$$

where

G_i = permanent load, such as D or H or T or P,
Q_l = principal variable load, such as L or S or W, or other load, taken in turn,
Q_i = any variable load such as L or S or W,
α_{G_i} = principal-load factor for the permanent load, G_i,
α_{Q_l} = principal-load factor for the principal variable load, Q_l,
α_{CQ_i} = companion-load factor for other variable loads, and

where the second term in Equation (2) is the principal variable load and the last term comprises the companion (expected) variable load or loads.

See Table A-3.

Load Combinations for Rare Loads or Situations

12. Where the load or situation is rare, the load combination is:

$$\Sigma\alpha_{G_i}G_i + A + \Sigma\alpha_{Q_i}Q_i \tag{3}$$

where

G_i = specified permanent load,
A = specified rare load due to earthquake, E, or other accidental load, such as that due to vehicle impact, I, or blast, B,
Q_i = specified companion variable load, such as L or S,
α_{G_i} = principal-load factor for the permanent load, G_i, and
α_{Q_i} = companion-load factor for companion variable loads, Q_i.

See Table A-3.

Table A-3
Load Factors for Rare Load Combinations

Loads		Principal-Load Factor, α_{G_i}	Companion-Load Factor, α_{Q_i}
Permanent loads, G_i	D, H, T, P	1.0(1)	—
Companion variable loads, Q_i	L	—	0.5(2)
	S	—	0.25
Rare loads, A	E, B, I	1.0	—

(1) Where the permanent load is D in combination with E, refer to NBC Sentence 4.1.3.2.(6).
(2) If E is the principal load, refer to NBC Tables 4.1.3.2.-A and -B.

Paragraph 25 provides guidance for resistance to fire—a rare event affecting the building structure.

Except as required by other regulations or standards, blast loading may be included in the design of a structure at the building owner's request. If such is the case, the owner will need to identify the design criteria, including the threat level and the desired performance level of structures and building elements, and the engineers can consult CSA S850, "Design and Assessment of Buildings

Subjected to Blast Loads," which is a performance-based standard providing criteria for the analysis and design of new buildings and the assessment of existing buildings. This standard does not provide information on how to conduct a risk assessment and is not applicable to biological or chemical threats, cratering, electromagnetic pulse (EMP), ground shock, fragmentation loads, induced fire, perimeter security, radiation, or thermal effects of explosions.

Load Combinations for Strength and Stability

13. The load combinations given in NBC Table 4.1.3.2.-A are simplified versions of Equations (2) and (3) and are based on reliability analyses.[3] They are applicable to most buildings and to structural systems within the scope of the standards currently listed in NBC Section 4.3. Use of these load combinations on structural members whose design is governed by and carried out for load combinations 2 to 4 with the companion load results in, on average, a probability of failure that is the same as that for cases where the design is controlled by D + L. Load T listed in NBC Article 4.1.2.1. and in Equation (2) is not included in NBC Tables 4.1.3.2.-A and -B because research and experience show that, except for secondary moments due to prestressing, this load is not likely to affect the strength and stability of structural systems that have ductility and redundancy. If a structural system lacking these properties is used, then load combinations should be determined from Equation (2) or (3) with the appropriate fractile of T included. Due to the very short duration of some specified loads, the probability of their simultaneous occurrence is extremely small. Thus, according to load combinations 5 and 6 of NBC Tables 4.1.3.2.-A and -B, respectively, earthquake load is not considered simultaneously with wind load.

Load Factors in NBC Table 4.1.3.2.-A

14. Applying the principal load factor to one of the specified loads accounts for variability of the load and load patterns, bias in the relationship between the nominal load and the expected value of the load for the event being considered, and normally accepted modelling approximations in the structural analysis. The principal load factors are determined based on these considerations as well as on experience gained from buildings built in accordance with previous editions of the NBC. The principal load factors for rare loads such as earthquake are taken equal to 1.0 because of their low annual probability of occurrence. The level of performance for rare loads allows building damage while maintaining life safety. The principal load factors are taken equal to 1.5 for live load due to use and occupancy and for earth pressure, 1.5 for snow load, 1.4 for wind load, and 1.25 for liquids whose depth is controlled.

15. The dead load factor of 1.25 accounts for the systematic and random variation of the dead load but is insufficient to accommodate dead load changes due to construction substitutions or subsequent alterations. Designs should anticipate and account for reasonable increases in the dead load of architectural or mechanical superimposed dead loads, of cast-in-place toppings and cover slabs that may be sensitive to the camber and deflection of the supporting members, and due to the addition of roofing or other materials during the life of the structure. For soil, superimposed earth, plants and trees, the dead load factor is increased to 1.5 but may be reduced in accordance with NBC Sentence 4.1.3.2.(8). Load combination 1 given in NBC Table 4.1.3.2.-A ensures the reliability of structural components that are dominated by dead load.[3]

16. The load factors for the serviceability limit states are taken equal to 1.0 or, for companion loads, less than 1.0 because of the less serious consequences of failure and because design criteria for serviceability are more subjective than for strength and stability.

17. The principal- and companion-load factors specified in the factored load combinations in NBC Table 4.1.3.2.-A are based on reliability analyses[3] calibrated according to past experience using previous editions of the NBC. The determination of load factors was carried out in two phases: the first phase involved determining values that provide uniform values of the reliability indices for a range of ratios and load types; in the second phase, the factors were reviewed and adjusted where necessary to reduce major inconsistencies with former practices.

18. The resistance factors given in the referenced material design standards, which take into account the variability of material properties, dimensions and workmanship, the type of failure (e.g. gradual versus sudden) and uncertainty in modelling resistance, have been developed for use with the load factors in the NBC to arrive at an acceptable level of safety, typically specified by a desired target reliability index.

Overturning, Uplift, Sliding and Stress Reversal

19. Counteracting loads, such as dead load, prevent overturning, uplift, the sliding of structures as a whole, and stress reversal or force reduction in structural members, which results in a reduced resistance due to, for example, the buckling of truss diagonals or the reduced flexural resistance of concrete columns. In such cases, counteracting loads that act to resist failure and deviations, which decrease rather than increase the dead load, are critical.[4][5] For load combinations 2 to 5 in NBC Table 4.1.3.2.-A and 1 to 6 in NBC Table 4.1.3.2.-B, counteracting variable loads are therefore taken equal to zero; the load factor for counteracting dead load (actually, a resistance factor) is taken equal to 0.9 in load combinations 2 to 4 in NBC Table 4.1.3.2.-A and 1 to 5 in NBC Table 4.1.3.2.-B; and the load factor for counteracting dead load is taken equal to 1.0 in load combination 5 in NBC Table 4.1.3.2.-A and load combination 6 in NBC Table 4.1.3.2.-B. The dead load factor was increased from 0.85 to 0.9 in the NBC 2005 because some gravity live load is expected to occur in most buildings. The dead load factor of 1.0 in the load combination that accounts for earthquake allows for the greater uncertainty in the magnitude of earthquake load and the reduced level of performance permitted with respect to building damage.

20. When assessing overturning, designers should consider the following:
- (1) the reaction of the foundation material is at such a distance from the toe of the building structure so as to generate the necessary reaction, and
- (2) the dead load acts through the centre of gravity of the deflected structure.

Cantilever Retaining Walls

21. When assessing overturning of cantilever retaining walls, designers should consider the following:
- (1) the reaction of the foundation material is at such a distance from the toe of the retaining wall so as to generate the necessary reaction, and
- (2) the dead load acts through the centre of gravity of the deflected retaining wall.

22. CSA S6, "Canadian Highway Bridge Design Code," and the Canadian Foundation Engineering Manual[6] provide additional guidance on the design of cantilever retaining walls.

Full and Partial Loading

23. Full and partial loading considerations are required as per NBC Article 4.1.5.3. for live load due to use and occupancy, NBC Article 4.1.6.3. for snow load, and NBC Article 4.1.7.9. for wind load. To achieve an acceptable reliability, pattern loading requirements for live or snow load should be considered in conjunction with the dead load multiplied either by 1.25 on all spans or 0.9 on all spans, whichever produces the most critical effect.

Load Combinations for Industrial Buildings

24. For building structures subjected to unusual loads not specified in NBC Part 4, for example those where liquids are stored, the load combinations given in NBC Tables 4.1.3.2.-A and -B may not apply. For guidance on industrial buildings with crane operations, see CSA S16, "Design of Steel Structures."

Load Combination for Determination of Fire Resistance

25. A rare event such as a fire can result in a temporary change of material properties causing large structural deformation and the potential for collapse. Structural fire resistance is defined as the time to structural failure when the structure is subjected to a standard fire. Structural fire resistance has traditionally been based on standardized fire tests and those mentioned in Appendix D of Division B of the NBC. When alternative measures, such as rational design, are used to design for fire resistance, an appropriate time–temperature fire curve should be used for calculations that take into account the forces in the structure due to the applied loads, including those having developed due to the high temperatures, and the properties of materials at high temperatures. The following load combination based on Equation (3) is recommended for an accidental event as the alternative measure design procedure for fire resistance:[7]

$$D + T_S + (\alpha L \text{ or } 0.25S) \qquad (4)$$

where $\alpha = 1.0$ for storage areas, equipment areas and service rooms, and 0.5 for other occupancies, and T_S can be taken equal to zero for statically determinate structures.

The appropriate use of Equation (4) requires expert knowledge in rational design for fire resistance. Where such expertise and/or the time–temperature fire curve are not available, the fire resistance requirements stated in the NBC should be applied.

Loads and Load Combinations for Fatigue Limit State

26. The variable of overriding importance in structural fatigue that dominates the propagation of cracks in metal components is the range of stress. The stress range to be used in the design is dictated by the variable loads. Because many cycles of load are required to cause fatigue, specified live loads that occur with reasonable frequency are used in design and not the extreme factored loads that have a very small probability of occurrence in the life of the structure. CSA S16 and CAN/CSA-S157/S157.1, "Strength Design in Aluminum/Commentary on CSA S157-05, Strength Design in Aluminum," only require detailed design against fatigue for more than 20,000 repetitions of load, except for the unusual case of fatigue-sensitive details with high stress ranges. Moreover, because cracks propagate in tensile stress fields only, the presence of a compressive stress field due to dead load, if sufficiently large, may obviate the development of cracks. In these circumstances, the accompanying dead load stresses should be assessed as discussed in CSA S16. Environmental loads, such as snow or wind loads, do not generally have a nearly sufficient number of cycles to be considered for the fatigue limit state. A possible exception is wind-excited vibrations such as vortex shedding and aerobics (see Commentary D). CSA S16 also addresses the concept of distortion-induced fatigue.

Loads and Load Combinations for the Serviceability Limit States

27. Loads and load combinations for serviceability calculations depend very much on the serviceability limit state under consideration and on the properties of structural materials (e.g., creep and cracking in concrete). Table A-4 provides guidance on the loads to be considered for serviceability criteria contained in NBC Part 4, in material design standards referenced in NBC Section 4.3., and in Commentaries D, E, I, J and K. Table A-4 also provides guidance on the load combinations of factored service loads (based on Equations (2) and (3)) to be considered depending on the limit state. Loads acting in combination do not need to be considered for vibration serviceability calculations. On the other hand, damage to the building structure or envelope may require the consideration of many loads in combination, particularly if the components are brittle.

Table A-4
Loads and Load Combinations for Serviceability[1]

Limit State	Structural Parameter	Loads	Load Combinations	References
Vibration serviceability	Acceleration	L_C,[2] W_C[2]	L_C[2] or W_C[2]	Commentary D Commentary I CSA O86 CSA S16
Operation of moving equipment	Deflection: Long-term Short-term	 D, H, T_P,[3] P L	 D + H + T_P[3] + P L	 CSA S16
Damage to non-structural components	Displacement: Long-term Short-term	 T_P,[3] P L, S, W	 T_P[3] + P L + α[4]S or S + α[4]L or W	Commentary D Commentary E CSA A23.3 CSA O86 CSA S16 CSA S304
Damage to structural components	Stress, strain, crack width	D, H, L, S, W, T_P,[3] T_S[5]	D + H + L_P[3] + T_P[3] + [L or S or W or T_S[5]] + companion loads	CSA A23.3 CSA S304 Commentary E

(1) S and W include an importance factor for serviceability.
(2) Subscript C refers to the cyclic components of load effects (e.g., acceleration).
(3) T_P includes creep (or soil settlement) under D + H + L_P + P, where L_P is the sustained component of live load due to use and occupancy.

Table A-4 (Continued)

(4) The companion load factor, α, is usually assumed to be 0.5 for live load due to use and occupancy, except for storage occupancies, where it is assumed to be 1.0, and 0.5 for snow load.
(5) T_S is the short-term variable effect caused by imposed deformations due to variations in temperature or moisture content, or a combination thereof.

Load Combinations for Settlement and Deflection of the Building Structure

28. Table A-5 presents recommended simplified combinations of service loads to determine the settlement of foundations causing building damage, and the deflection of the building structure causing building damage or impeding the operation of equipment such as cranes or elevators. For cases 2 and 3 in Table A-5, deflection of the building structure causing building damage is the sum of the short-term deflection occurring after the attachment of non-structural building elements plus the long-term component of deflection due to D, H, T_P and P resulting from shrinkage or moisture change and creep of materials occurring after the attachment of non-structural building elements. Because of the approximations required, the long-term deflection resulting from shrinkage or moisture change and creep is usually taken into account by specific empirical deflection limits stated in the design standards listed in NBC Section 4.3.

Table A-5
Recommended Load Combinations for Serviceability Limit States Governed by Deflection

Case	Serviceability Parameter	Load Combinations
1	Differential settlement of foundations	$D + H + \alpha^{(1)}L + \alpha^{(1)}S^{(2)}$
2	Long-term deflection of building structure(3)	$D + H + T_P^{(4)} + P + \alpha^{(1)}L + \alpha^{(1)}S^{(2)}$
3	Short-term deflection of building structure(3)	$(L + \alpha^{(1)}S^{(2)})$ or $(S^{(2)} + \alpha^{(1)}L)$ or $W^{(2)}$

(1) The companion load factor, α, is usually assumed to be 0.2 to 0.5 for snow load and for live load due to use and occupancy, except for storage occupancies, where it is assumed to be 1.0.
(2) Importance factors 0.9 and 0.75 are applied in NBC Subsections 4.1.6. and 4.1.7. to determine service loads S and W.
(3) For deflection of the building structure causing building damage, see Paragraph 27.
(4) T_P includes deflection caused by long-term moisture changes in materials (e.g., shrinkage), while creep deflection is calculated using the applied loading $D + H + P + \alpha L + \alpha S$.

History of Limit States Design Provisions in the NBC

29. Limit states design, introduced into the NBC in 1975, was initially developed for steel structures, and then for concrete structures, which had been designed based on ultimate strength design prior to 1975. It was later developed for wood, cold-formed steel and masonry structures, then in 1983 for aluminum structures, and finally in 1995 for foundations. Allowable stress design has been gradually phased out as an alternative to limit states design for steel, concrete, wood, masonry and foundations, but is still used as the basis for some standards and specifications not directly referenced by NBC Section 4.3.

30. In the 2005 edition of the NBC, the main changes to the limit states requirements of NBC Section 4.1. were as follows:
- the adoption of the companion action format of limit states design for load combinations, which is used worldwide,
- the separation of load due to snow and rain, S, from live load due to use and occupancy, L,
- the consistent use of importance factors applied to snow, rain, wind and earthquake loads, including an importance factor for serviceability depending on the use and occupancy of the building, and
- the modification of the return period for snow, rain and wind loads from 1/30 years to 1/50 years, which is used worldwide.

The methods for determining loads are now harmonized. Snow, rain and wind loads are calculated using a single return period and varying importance factor, which brings them in line with the approach used for earthquake loads. All loads and effects are combined using the companion action format, which provides a clear set of load combinations with direct physical meaning. The separation of load due to snow and rain from load due to use and occupancy allows for a more logical determination of load factors and load combinations based on the variability of loads and the probability of them acting together. The base return period for snow and wind loads was increased from 30 years to 50 years to more closely match the expected service life of a building while maintaining the same target probability against failure as in the 1995 NBC. The 50-year return period is consistent with the approach taken by most other countries.

References

[1] M.K. Ravindra and N.C. Lind. Theory of Structural Code Optimization. Journal of the Structural Division, Proc. Am. Soc. Civ. Eng., Vol. 99, ST7, July 1973, pp. 1541-1553.

[2] C.J. Turkstra, Theory of Structural Safety. SM Study No. 2, Solid Mechanics Division, University of Waterloo, Waterloo, Ontario, 1970.

[3] F.M. Bartlett, H.P. Hong, and W. Zhou. Load factor calibration for the proposed 2005 edition of the National Building Code of Canada: Companion-action Load Combinations. Canadian Journal of Civil Engineering, Vol. 30, No. 2, April 2003, pp. 440-448.

[4] D.E. Allen, Safety Factors for Stress Reversal. Publications, International Association for Bridge and Structural Engineering, Vol. 29, No. 2, 1969, pp. 19-27.

[5] Report of the Committee of Inquiry into Collapse of Cooling Towers at Ferrybridge, Monday, 1 November 1965. Central Electricity Generating Board, London, 1966.

[6] Canadian Foundation Engineering Manual, 4th Edition. Canadian Geotechnical Society, Richmond, British Columbia, 2006.

[7] B. Ellingwood and R.B. Corotis, Load Combinations for Buildings Exposed to Fires. AISC Engineering Journal, 28(1), 1991, pp. 37-44.

Commentary B

Structural Integrity

Commentary B

Structural Integrity

1. The strength and stability of building structural systems is addressed in NBC Sentence 4.1.1.3.(1) and in specific requirements in NBC Part 4, and in the CSA material design standards referenced in NBC Section 4.3. This Commentary provides guidance on additional considerations regarding structural integrity as addressed in NBC Sentence 4.1.1.3.(1) and its explanatory Note.

2. Structural integrity is defined as the ability of the structure to absorb local failure without widespread collapse. For example, a cellular or frame arrangement of components that are well tied together in three dimensions has good structural integrity.

3. Building structures designed in accordance with the CSA design standards will usually have an adequate degree of structural integrity, which is generally achieved through detailing requirements for the connections between components. Situations where structural integrity may require special attention include medium-rise and high-rise building systems made of components of different materials, whose interconnection is not covered by existing CSA design standards, buildings outside the scope of existing CSA design standards, and buildings exposed to severe accidental loads such as vehicle impact or explosion. The following paragraphs provide guidance for such situations.

4. A significant number of failures—many of them progressive—occur during construction. The construction sequence should, therefore, be carefully planned and monitored to ensure that partially completed structural systems have sufficient strength, ductility and lateral stability to resist progressive collapse if a construction accident causes significant damage to a structural element or if local failure of a permanent or temporary structural element occurs.

Identification of Hazard

5. The hazard is the risk of widespread collapse with serious consequences that arises from local failure caused by accidental events not addressed by the loads specified in NBC Part 4. Key components that can be severely damaged by an accident with a significant probability of occurrence (i.e., approximately 10^{-4} per year or more) should therefore be identified, and measures should be taken to ensure adequate structural safety.[1]

Safety Measures

6. The occurrence of widespread collapse resulting from accidental events can be prevented through safety measures such as the following:

(a) Control of accidental events: Such measures include the erection of protective devices (e.g., curbs, guards) against vehicle impact, the inspection of key elements or ground conditions for deterioration during use, and blow-out panels to reduce explosion pressures.

(b) Local resistance: This consists of designing key members to resist accidental events.[2] Some major structural members, for example, are so strong that most accidental events are unlikely to cause serious structural damage. Ductility of the key members and of their connections to the structure can also provide substantial additional resistance to accidents not normally considered during design.

(c) Design of tie forces: Structural integrity can often be achieved indirectly by providing certain minimum criteria for vertical, horizontal and peripheral ties in buildings.[3][4][5]

(d) Alternate paths of support: Here it is assumed that the key member has failed, and the damaged building is checked to ensure that it can support the dead load plus a portion of the live load and wind load.
(e) Control of widespread collapse: This measure consists of dividing the structure into areas separated by planes of weakness, which will prevent a collapse in one area from propagating into adjacent areas (see Commentary C).

7. Any building system should be considered as a whole and effectively tied together in such a way as to not be sensitive to local accidental failure.

8. Additional information for specific building structural systems is contained in References [3] to [9]. Reference [6] contains additional references regarding concrete building systems.

References

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[2] DS 410 E:1983, Loads for the Design of Structures, Chapter 17 – Accidental Action. Dansk Ingeniorforening, Copenhagen, 1983.

[3] J.E. Breen, Developing Structural Integrity in Bearing Wall Buildings. PCI Journal. Vol. 25, No. 1, 1980, pp. 42-73.

[4] M. Fintel and G. Annamalai, Philosophy of Structural Integrity of Multistory Load-bearing Concrete Masonry Structures. Concrete International, Vol. 1, No. 5, 1979, pp. 27-35.

[5] I.J. Speyer, Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads. PCI Journal, Vol. 21, No. 2, 1976, pp. 18-51.

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[8] B.R. Ellingwood and E.V. Leyendecker, Approaches for Design Against Progressive Collapse. Journal of the Structural Division, Proc. Am. Soc. Civ. Eng., 1978, pp. 413-423.

[9] D.A. Taylor, Progressive Collapse. Can. J. Civ. Eng., Vol. 2, No. 4, 1975.

Commentary C

Structural Integrity of Firewalls

Commentary C

Structural Integrity of Firewalls

1. NBC Sentence 3.1.10.1.(1) requires that, where structural framing members are connected to or supported on a firewall and their fire-resistance rating is less than that required for the firewall, the connections and supports for such members must be designed so that the collapse of the framing members during a fire will not cause the collapse of the firewall. NBC Sentence 4.1.5.17.(1) requires that the firewall be designed to resist a factored lateral load of 0.5 kPa under fire conditions.

2. These requirements, along with others in NBC Subsection 3.1.10., form part of the general requirement that a fire not spread between compartments separated by a firewall within the required fire-resistance rating for that wall (4 h for high fire hazard occupancies and 2 h for other occupancies). To achieve this, the firewall must not be damaged to such an extent that it allows a fire to spread within these periods.

3. In order to meet the requirement for structural integrity of firewalls, the following loading conditions must be applied.

Lateral Loads on Firewalls

4. NBC Sentence 4.1.5.17.(1) requires that firewalls be designed for a factored lateral load of 0.5 kPa so that, during a fire, the firewall will not collapse due to the explosion of unburned gases, glancing blows from falling debris, the force and thermal shock of a firehose stream and wind pressure. If the structure exposed to the fire has less fire resistance than that required for the firewall, it is assumed to have failed and therefore to provide no lateral support to the firewall.

5. NBC Sentence 4.1.5.17.(1) also requires that the firewall be designed in accordance with the typical structural requirements applicable to interior walls with regard to wind and earthquake, as well as pounding damage.

6. The building structure, including the firewall, should also be designed to provide structural integrity in accordance with the recommendations of Commentary B.

Thermal Effects

7. The thermal expansion of a structure exposed to fire must not damage the firewall as this would allow the premature spread of fire through the wall.

8. To assess the potential for such damage, the thermal expansion of the structure should be estimated based on a 500°C temperature increase in combination with the thermal coefficients given in Table E-1 of Commentary E. The expansion of the structure toward the firewall can be assumed to begin at a vertical plane in the fire compartment at 20 m from the firewall or half the width of the fire compartment, whichever is less.

9. In assessing thermal effects, attention should be given to the effect that distortion of the firewall due to temperature differential through the wall has on the stability of the firewall.

10. If thermal movements are sufficient to damage the firewall, either adequate clearances should be provided or the firewall and structure on both sides should be detailed to prevent wall damage.

Design Approaches

11. Some design approaches that satisfy the general requirements for the structural integrity of firewalls are described in Paragraphs 12 to 15.

Double Firewall (NBC 3.1.10.1.(2))

12. The structure on each side is tied to a separate firewall in such a way that, when the structure exposed to fire fails, only one firewall will collapse without damaging the remaining firewall. A schematic example is shown in Figure C-1. Each wall should have at least half the total required fire-resistance rating. The separation between the walls must satisfy the requirements regarding thermal expansion stated in Paragraphs 7 to 10 and those regarding earthquakes stated in Commentary J.

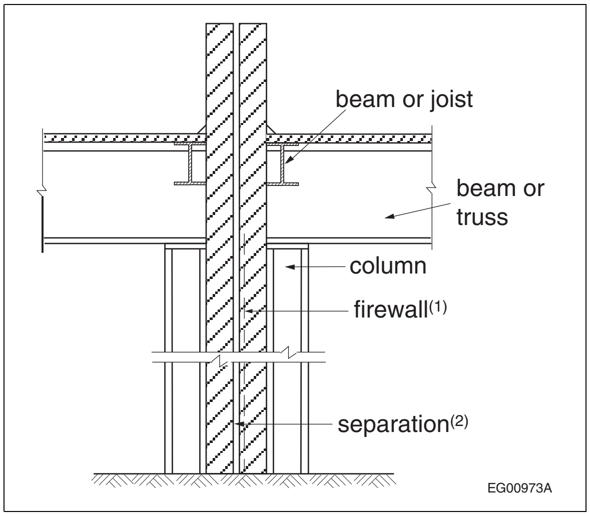

Figure C-1
Schematic example of a double firewall

Notes to Figure C-1:

(1) Each firewall must be tied to the adjacent structure in accordance with Paragraph 12 and reinforced in accordance with Paragraphs 4 and 5.

(2) Firewalls must be separated in accordance with Paragraph 10.

Cantilever Firewall

13. In this design approach, the structure on either side is not connected to the firewall, so that the collapse of the structure exposed to fire does not cause the collapse of the firewall. A schematic example is shown in Figure C-2. Reinforcement of the cantilever wall and foundations against overturning will generally be required to resist the lateral loads specified in NBC Sentence 4.1.5.17.(1). Pilasters will frequently be needed to provide this requisite lateral load capacity.

Tied Firewall

14. In this approach, the structure on each side of the firewall provides lateral support to the firewall and is tied together in such a way that lateral forces resulting from the collapse of the structure exposed to fire are resisted by the structural framework on the other side of the firewall. Lateral forces are recommended in Paragraphs 4 and 5; suitable provisions must be made to transmit these forces to members on opposite sides of the firewall. A schematic example is shown in Figure C-3.

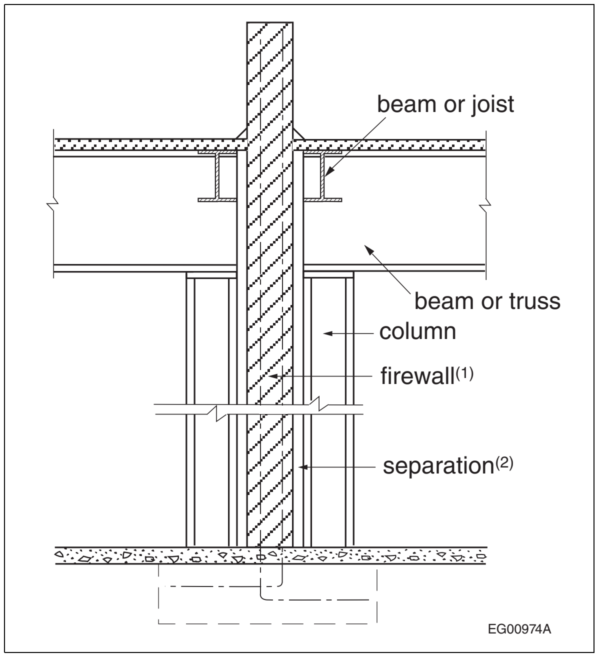

Figure C-2
Schematic example of a cantilever firewall

Notes to Figure C-2:
(1) The firewall is not tied to the structure and is designed as a cantilever from the foundation, with reinforcement and pilasters in accordance with Paragraphs 4, 5, 10 and 13.
(2) Separation may be required in accordance with Paragraph 10.

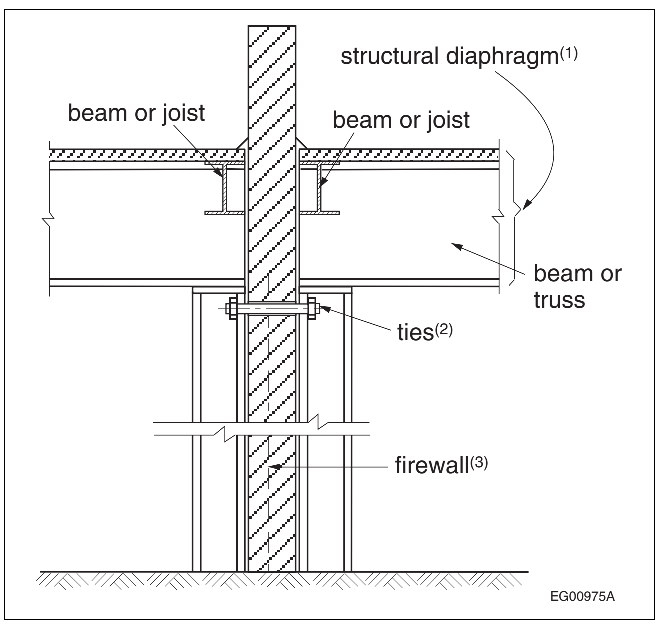

Figure C-3
Schematic example of a tied firewall

Notes to Figure C-3:
(1) Structural diaphragm resistance may be required in accordance with Paragraphs 12, 16 and 17.
(2) Ties must be located and detailed in accordance with Paragraphs 12, 16 and 17.
(3) The firewall must be reinforced and detailed in accordance with Paragraphs 4, 5 and 10.

Weak-Link Connections

15. Here, structural components are supported by the firewall in such a way that a failing structure will collapse without causing the firewall to be severely damaged. As with a tied firewall, the structure may also provide lateral support to the firewall. If a weak link is provided on each side of the firewall, the link on the fire side will break away while the link on the non-fire side will not. This approach has traditionally been used in timber construction, where timber beams or joists bear without anchors into pockets of firewalls and can twist free when they collapse.[1][2] Figure C-4 shows a more recent technique for a weak-link connection to a block firewall. If this technique is used, care must be taken to provide adequate anchorage to resist wind uplift and earthquake.

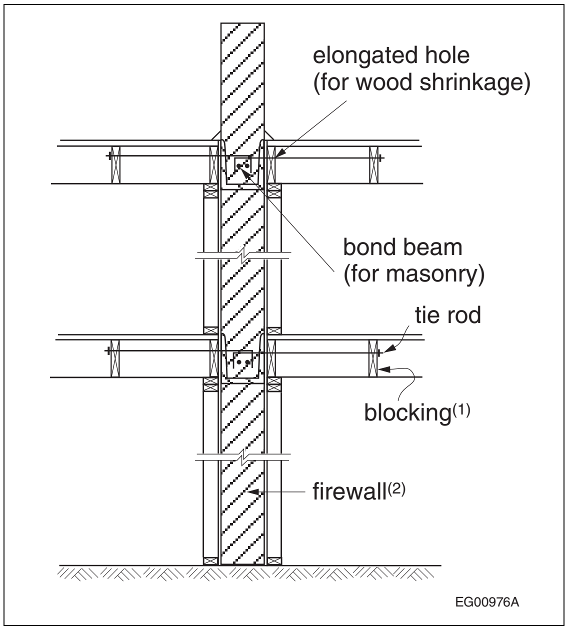

Figure C-4
Example of a weak-link connection used in wood-frame construction

Notes to Figure C-4:
(1) The blocking connection to the wood frame must be detailed to act as a weak link in accordance with Paragraph 15.
(2) The firewall must be reinforced and detailed in accordance with Paragraphs 4, 5 and 10.

Tied Firewalls: Horizontal Forces from Collapsing Structure

16. Where a structure with a lower fire resistance than that required for the firewall is tied through the firewall to the structure on the other side of the firewall, the supporting structure and the ties should be designed for a factored horizontal force equal to $wBL^2/8S$, where w is the dead weight plus 25% of the specified snow load, B is the distance between the ties, L is the span of the collapsing structure between columns perpendicular to the wall, and S is the sag of the collapsing structure, which is assumed to be 0.07 L for steel open-web beams and 0.09 L for steel solid-web beams. The supporting structure should be capable of resisting the forces recommended for the ties within a 10 m length of firewall; the other ties are assumed to carry no force (see Figure C-5). The factored resistance of the ties should include a reduction factor of 0.5 to account for a reduced yield strength at high temperature.

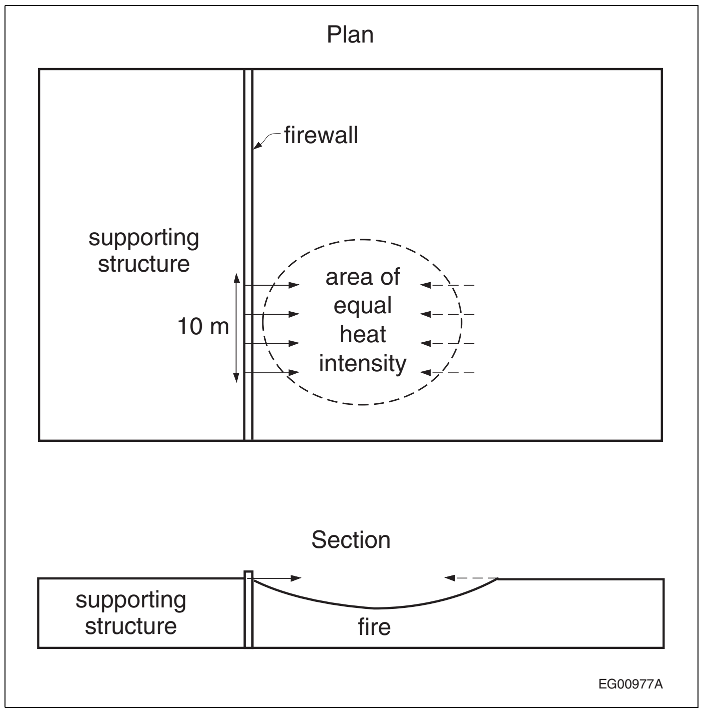

Figure C-5
Horizontal forces on a tied firewall

17. Alternatively, if the firewall is located so that the roof structure has the same resistance to horizontal forces on either side of the firewall (e.g., the firewall is located mid-way between end walls or expansion joints of a structurally symmetric building), only the ties need to be designed for the factored horizontal force $wBL^2/8S$.

References

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Commentary D

Deflection and Vibration Criteria for Serviceability and Fatigue Limit States

Commentary D

Deflection and Vibration Criteria for Serviceability and Fatigue Limit States

Notable Change in this Commentary

- Updating of Table D-1 to align with CSA design standards

General

1. The advent of stronger materials, lighter construction, more rigid cladding, smaller damping, longer spans and more accurate strength calculations, which take into account the interaction of building components, means that excessive deflection and vibration now have a greater influence on structural design than before. In the past, building codes controlled excessive deflection and vibration by limiting the member deflection under specified load to some ratio of the span L, for example, L:360 (for cantilevers, L may be taken as twice the length of the cantilever). This widely used criterion dates back to the mid-nineteenth century. To help designers, this Commentary discusses and provides guidance on the problems associated with excessive deflection and excessive vibration.

Deflection

2. Excessive structural deflection can create a variety of problems: cracks in or crushing of non-structural components such as partitions, lack of fit for doors or windows, out-of-plumb walls, end rotation resulting in damage due to eccentric forces, unsightly droopiness, and ponding of water. Cracks, besides being unsightly, may transmit unwanted sound through partitions, or water and cold air through the building envelope, and thus promote material deterioration. Control of cracking in structural concrete is covered separately in CSA A23.3, "Design of Concrete Structures."

3. A number of alternative design solutions can prevent problems caused by excessive deflection. Partition cracking, for example, can be avoided either by making the supporting structure stiff enough or by providing flexible joints in the partitions. Similarly, to avoid cracking, plastered ceilings should be hung from the floor structure, not rigidly attached to it.

4. Table D-1 summarizes the deflection criteria contained in NBC Part 9 and in the design standards referenced in NBC Part 4. These criteria apply to conventional forms of construction under conventional conditions of use. The most severe deflection requirement, 1:720, which applies to members supporting components susceptible to cracking,[1] may not prevent cracking.[2] For new or unusual situations involving concrete structures, more detailed deflection criteria are recommended in Reference [3]; case histories of damage due to excessive deflection—including differential settlement, shrinkage, creep and temperature movements—are presented in References [4] to [7].

Table D-1
Summary of Maximum Deflection/Span Ratios in NBC Part 9 and Referenced CSA Standards(1)

Building Component	Referenced Document				
	CSA O86 Wood	CSA A23.3 Concrete	CSA S16 Structural Steel	CSA S304 Masonry(2)	NBC Part 9 of Division B
Roof or floor members supporting components susceptible to cracking	—	1:480(3) or 1:240(3)	1:360(4)	—	1:360
Floor members not supporting plastered components susceptible to cracking	1:360(5) or 1:180(5)	1:360(6)	1:300(4)	1:480(7)	1:360
Roof members not supporting plastered ceilings, etc.	1:360(5) or 1:180(5)	1:180(6)	1:300(4)(8)	—	1:180(9) or 1:240
Wall members	1:180 or 1:360(10)	—	—	1:180 to 1:360(7)	—

(1) Deflection under live, snow or wind load only, unless otherwise noted.
(2) Structural support of masonry:
(a) lateral support of masonry walls – 1:240 to 1:600 depending on type, material and direction of wall flexure;
(b) vertical support of masonry walls – 1:480 ≤ 20 mm; and
(c) for glass block walls – 1:600.
(3) Deflection that occurs after the attachment of non-structural components, including creep deflection due to sustained load plus immediate deflection due to live or snow load. 1:240 applies when non-structural components are not likely to be damaged by large deflections.
(4) Special limits are given for steel roof structures (1:180 to 1:240 depending on roofing) and craneways (1:600 to 1:800 depending on crane capacity) on industrial buildings.
(5) 1:180 will control immediate deflection under total serviceability loads, except for members that have been cambered for the dead load deflection, in which case the additional deflection due to live, snow and wind loads must not exceed 1/180 of the span. 1:360 will control elastic deflection under long-term loads that exceed 50% of the total serviceability loads. A special deflection criteria is recommended to control ponding on flat roofs.
(6) Immediate deflection due to live, snow or wind load.
(7) Reinforced masonry walls and columns – 1:180 to 1:360 wind deflection; reinforced masonry beams – 1:480.
(8) See Commentary H for a warning on ponding.
(9) 1:180 applies if there is no ceiling.
(10) 1:360 is recommended to control damage to masonry veneer due to wind deflection of wood stud walls. See Annex A.5.4.2 of CSA O86.

Floor Vibration

5. Two types of vibration problems arise in building construction: continuous vibrations and transient vibrations. Continuous vibration arises due to the cyclic forces of machinery or certain human activities such as dancing; this vibration can be considerably amplified when the cyclic forces are synchronized with a building frequency—a condition called resonance. Transient vibration is caused by persons jumping or other impact (e.g., dropping of weights in a health club, vehicle impact in a parking garage), and decay at a rate that depends on the available damping.

Floor Vibration Due to Walking

6. The vibration of floor systems due to walking may cause annoyance to occupants. The deflection limits in Table D-1 have, in the past, been used in an attempt to control such vibration but, because of the unsatisfactory vibration performance of buildings designed to these limits, they have been replaced in recent years by new criteria based on the dynamic vibration of building structures (see ISO 10137, “Bases for Design of Structures – Serviceability of Buildings and Walkways Against Vibration,” and Reference [8]). Recommended criteria to control vibration due to walking are contained in Reference [9] for steel construction and in Reference [10] for all structural materials, including light-frame construction. A concentrated-load deflection criterion based on experience[11] was used in the NBC Part 9 of Division B span tables to address walking vibration in light-frame floors.

7. An unusual form of vibration present in pedestrian bridges is lateral sway vibration due to resonance caused by heavy pedestrian traffic. Such vibration could occur in a laterally flexible structure used for heavy pedestrian traffic such as a suspended walkway. For guidance on the subject, see the section titled Serviceability Limit States in CSA S6.1, “Commentary on CSA S6-14, Canadian Highway Bridge Design Code.”

Floor Vibration Due to Machinery

8. The undesirable effects of continuous vibration caused by machines can be minimized by special design provisions,[8][9] such as locating machinery away from sensitive occupancies, vibration isolation, or alteration of the frequency of the structure.

Floor Vibration Due to Rhythmic Activities

9. NBC Sentence 4.1.3.6.(2) requires that a dynamic analysis be carried out for floor structures (including footbridges) supporting assembly occupancies whose fundamental vibration frequency is less than 6 Hz. This requirement was introduced because of vibration problems with long-span floor structures used for rhythmic activities.[12] to [17] The following paragraphs provide guidance for designers on how to carry out a dynamic analysis for such cases, and recommend criteria to limit floor vibrations during rhythmic activities to levels acceptable for human occupancy of the building.

Dynamic Loading and Response Due to Rhythmic Activities

10. Dancing, foot stamping, jumping exercises and marching are rhythmic activities that create periodic forces with step frequency (e.g., beat of music) in the range of 1 to 4 Hz. For rhythmic activities involving a group of people, the most critical range is 2 to 2.75 Hz. Typical loading cases are shown in Figure D-1. For rhythmic activities, such as dancing, the periodic forces can be approximated by a sinusoidal dynamic load causing vibration at the step frequency, f_s. In the case of jumping exercises, however, the periodic forces shown in Figure D-1 can also create significant sinusoidal load at double the step frequency, $2f_s$, and some sinusoidal load at triple the step frequency, $3f_s$. For any harmonic multiple, i, of the step frequency, the forcing frequency is equal to if_s. The sinusoidal dynamic load applied to the floor for any harmonic can therefore be represented by $\alpha_i w_p \sin 2\pi i f_s t$, where α_i is a dynamic coefficient that varies depending on the activity, w_p is the effective weight of participants per unit area in kPa, if_s is the forcing frequency, and t is time. Table D-2 recommends values of the forcing frequencies, if_s, of the dynamic load based on an estimation of density and weight of participants, w_p, and of the dynamic coefficient, α_i, for typical rhythmic events. These values are based on References [15] and [16] and on recent experience. If the forcing frequency, if_s, is smaller than the fundamental natural frequency of the floor structure (the floor frequency), f_n, the dynamic load has the same effect (e.g., displacement, member force) as a static load of the same magnitude, but if the forcing frequency approaches the floor frequency, the dynamic effect increases with each cycle of vibration to a maximum (see Figure D-2) whose ratio to the static effect is given by

$$\rho = 1/\sqrt{\left[1 - \left(\frac{f}{f_n}\right)^2\right]^2 + \left(\frac{2\beta f}{f_n}\right)^2} \tag{1}$$

where the forcing frequency, f, equals if_s, and β is the damping ratio.[8] If a floor has many people on it, the damping ratio, β, is about 0.06 for a concrete floor and a steel floor with a concrete deck, and 0.12 for a light-frame floor; the damping ratio is about half these values if a floor has few people on it. Damping ratios vary from these suggested values, depending on the influence of non-structural components such as partitions. The dynamic amplification factor, ρ, is shown in Figure D-3 as a function of f/f_n. When multiplied by the cyclic peak dynamic load, $\alpha_i w_p$, the product, $\rho\alpha_i w_p$, is a static load (called the equivalent static load) whose effect is the same as that of the cyclic dynamic load, $\alpha_i w_p \sin 2\pi f t$.

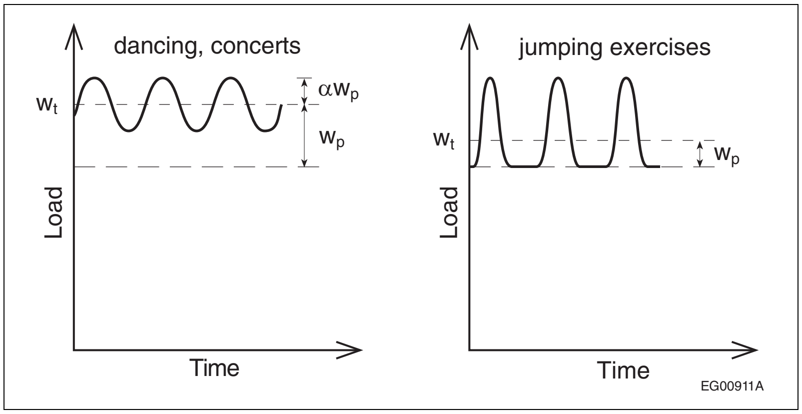

Figure D-1
Load during rhythmic event

Table D-2
Recommended Loading Function for Rhythmic Events

Activity Property	Activity		
	Dancing	Lively Concert(1) or Sports Event	Aerobics
Weight of participants,(2) w_p, kPa	0.6 (2.5 m²/person)	1.5 (0.5 m²/person)	0.2 (3.5 m²/person)
First harmonic,(3) α_1 (forcing frequency, f_s)	0.5 (1.5 to 2.7 Hz)	0.25 (1.5 to 2.7 Hz)	1.5 (2 to 2.75 Hz)
Second harmonic,(3) α_2 (forcing frequency, $2f_s$)	0.05 (3 to 5 Hz)	0.05 (3 to 5 Hz)	0.6 (4 to 5.5 Hz)
Third harmonic,(3) α_3 (forcing frequency, $3f_s$)	–	–	0.1 (6 to 8.25 Hz)

(1) Values given are for concerts where there is fixed seating. For rock concerts at which seating is not provided, α_1 = 0.40 and α_2 = 0.15.
(2) Weight of participants is uniformly distributed over activity area. For long-span floors where dancing occurs only on part of the span, the effective uniformly distributed weight over the whole span may be reduced accordingly.
(3) Values of the dynamic coefficient for the i'th harmonic, α_i, are based on commonly encountered events involving a minimum of 20 persons.

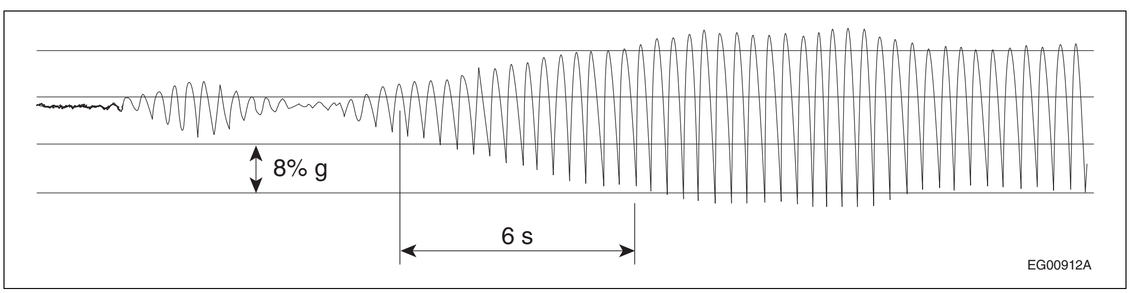

Figure D-2
Resonance during a rock concert (precast stands, f_n = 2.6 Hz)

11. The floor frequency, f_n , should be determined from the dynamic properties of the floor structure, taking into account the flexibility of supports. This can best be carried out using reliable dynamic FEM software. An approximate determination for simply supported joists or beams on girders supported by columns is obtained from

$$f_n = 18/\sqrt{\Delta} \tag{2}$$

where Δ is the deflection of the floor structure in mm, which can be conservatively approximated by

$$\Delta = \Delta_j + \Delta_g + \Delta_c \tag{3}$$

where

Δ_j = the elastic deflection of the joist or beam due to bending and shear, in mm,
Δ_g = the elastic deflection of the girder due to bending and shear, in mm, and
Δ_c = the elastic shortening of the column due to axial strain, in mm.

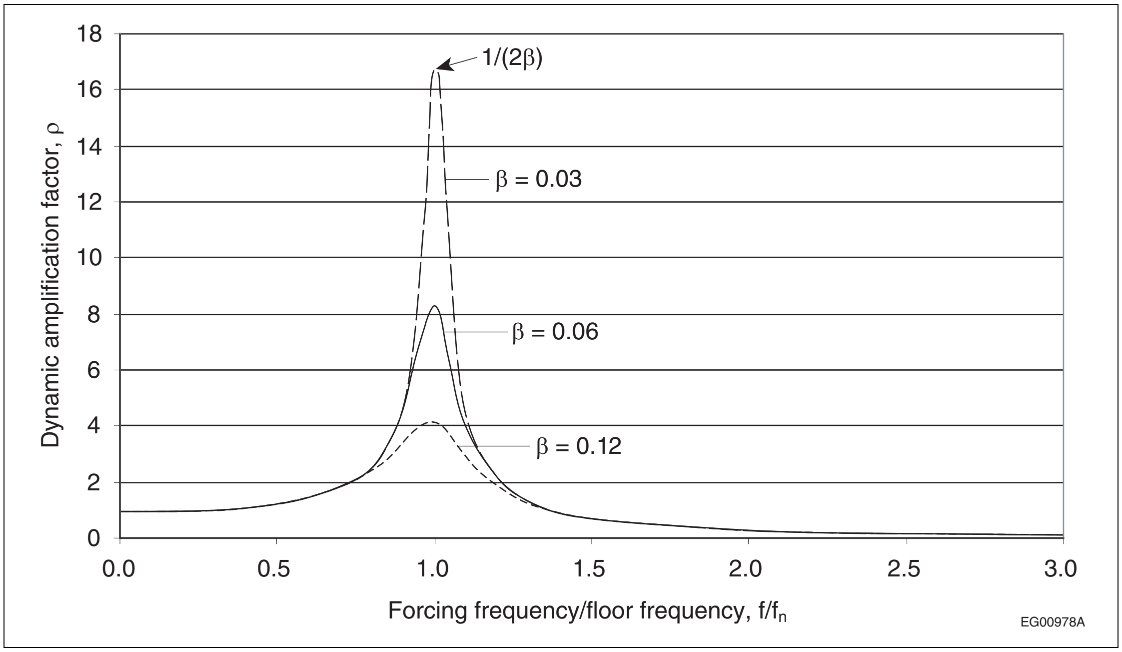

Figure D-3
Dynamic amplification factor, Equation (1)

Each deflection, Δ, is due to the total weight supported by the member, including people, and is relative to its supports. Both supports are considered and the most flexible one is used. In the case of joists, beams and girders that are continuous over supports, the elastic deflection, Δ_j or Δ_g, should be determined by assuming that adjacent spans deflect in opposite directions with no change in slope over the supports and that the weight supported by each span always acts in the direction of deflection.

Human Reaction

12. Floor vibration due to rhythmic activities is much more likely to annoy people than to cause overloading or fatigue. An acceptable level of vertical vibration depends very strongly on the activity of the people who feel the vibration. People in offices or residences become annoyed when accelerations from continuous vibration exceed approximately 0.5% gravity, whereas people participating in rhythmic activities will accept considerably greater than 10% gravity. People such as diners who share a floor structure with dancing will accept approximately 2% gravity. When a floor bay where rhythmic activities are going on is shared with a more sensitive occupancy, then the limit should be based on that occupancy. Other factors besides occupancy affect the acceptability of vibration, in particular the remoteness of the source of vibration from the people affected. For this reason, a range of acceleration limits for different occupancies is recommended in Table D-3. The limit of 4 to 7% gravity given in Table D-3 for a rhythmic activity area in an office or residential building is intended to control floor vibration in other areas of the building containing sensitive occupancies. The limit of 10 to 18% gravity for stadia containing no sensitive occupancies is based on testing[18][19] and feedback from experience.

Table D-3
Recommended Acceleration Limits for Vibrations due to Rhythmic Activities

Occupancies Affected by the Vibration	Acceleration Limit, % gravity
Office and residential	0.4 to 0.7
Dining and weightlifting	1.5 to 2.5
Rhythmic activity area	
in an office or residential building	4 to 7
in a stadium or arena	10 to 18

13. The maximum acceleration, a_{pi}, of a floor structure during a rhythmic event for each harmonic multiple, i, of the step frequency, f_s, can be determined from[15]

$$a_{pi}/g = \frac{1.3\alpha_i w_p/w_t}{\sqrt{\left[\left(\frac{f_n}{if_s}\right)^2 - 1\right]^2 + \left(\frac{2\beta f_n}{if_s}\right)^2}} \tag{4}$$

where the variables are defined in Paragraphs 10 and 14. The effective maximum acceleration for all harmonics, a_{max}, is obtained from[16]

$$a_{max} = \left[\sum a_{pi}^{1.5}\right]^{2/3} \tag{5}$$

14. If a floor frequency corresponds to a harmonic forcing frequency, resonance will occur and the accelerations during a rhythmic event will become very large—usually greater than the limit recommended in Table D-3. The floor frequency, f_n, should generally be greater than the highest significant harmonic forcing frequency, if_s. The following criterion, determined by inverting Equation (4) for sinusoidal loading,[15] is recommended:

$$\frac{f_n}{if_s} \geq \sqrt{1 + \frac{K}{a_o/g}\left(\frac{\alpha_i w_p}{w_t}\right)} \tag{6}$$

where
- a_o/g = acceleration limit as a ratio of the acceleration due to gravity,
- K = 1.3 for sinusoidal loading (from Equation (4)),
  = 2.0[16] for jumping exercises (3 harmonics combined), and
  = 1.7[10] for other rhythmic activities noted in Table D-2 (2 harmonics combined),
- w_t = total weight supported, in kPa, and
- $\alpha_i w_p$ = see Paragraph 10.

15. Table D-4 contains examples of the application of Equation (6) to typical floor structures using the acceleration limits recommended in Table D-3. A simple conservative procedure for the analysis of floor vibration is to compare the floor frequency calculated in accordance with Paragraph 11 with the minimum frequency for acceptable performance given in Table D-4. If the minimum is not met, it is recommended to use a more direct calculation of floor properties and performance, as shown in the example presented in Paragraphs 16 to 18.

Table D-4
Minimum Floor Frequency Based on Equation (6)[1]

Activity	Construction		
	Heavy Floor 5 kPa	Medium Floor 2.5 kPa	Light Floor 1 kPa
	Minimum Floor Frequency, Hz		
Dancing and dining ($a_o/g = 0.02$, $if_s = 2.7$ Hz, $w_p = 0.6$ kPa)	6.5	8.0	11.0
Lively concerts[2] or sports events ($a_o/g = 0.05$, $if_s = 5$ Hz, $w_p = 1.5$ kPa)	6.0	6.5	7.5
Aerobics only ($a_o/g = 0.06$, $if_s = 8.25$ Hz,[3] $w_p = 0.2$ kPa)	9.0	9.5	12.0
Aerobics and weightlifting ($a_o/g = 0.02$, $if_s = 8.25$ Hz,[3] $w_p = 0.12$ kPa)	9.0	11.0	15.0

(1) Equation (6) is applied to all harmonics (1, 2 or 3) but the governing harmonic is used. In some cases, however, damping × mass is sufficient to reduce high-harmonic resonance to an acceptable level.
(2) Assumes fixed seating (see Note (1) of Table D-2).
(3) Sometimes governed by second harmonic, f = 5.5 Hz.

Example

16. A 30 m by 50 m ballroom with a floor weight of 5 kPa is to be used for dining and dancing (see Figure D-4). The floor structure consists of a concrete deck on steel trusses of 30 m span supported by steel girders of 5 m span on one-storey columns; the primary flexibility of the floor structure is provided by the trusses. Table D-4 indicates a minimum natural frequency of 6.5 Hz for satisfactory performance of the floor. In accordance with Equation (2), this natural frequency corresponds to a deflection of the floor structure, Δ, of only 7.7 mm (span/3900), which is very difficult for a 30-m span to achieve.

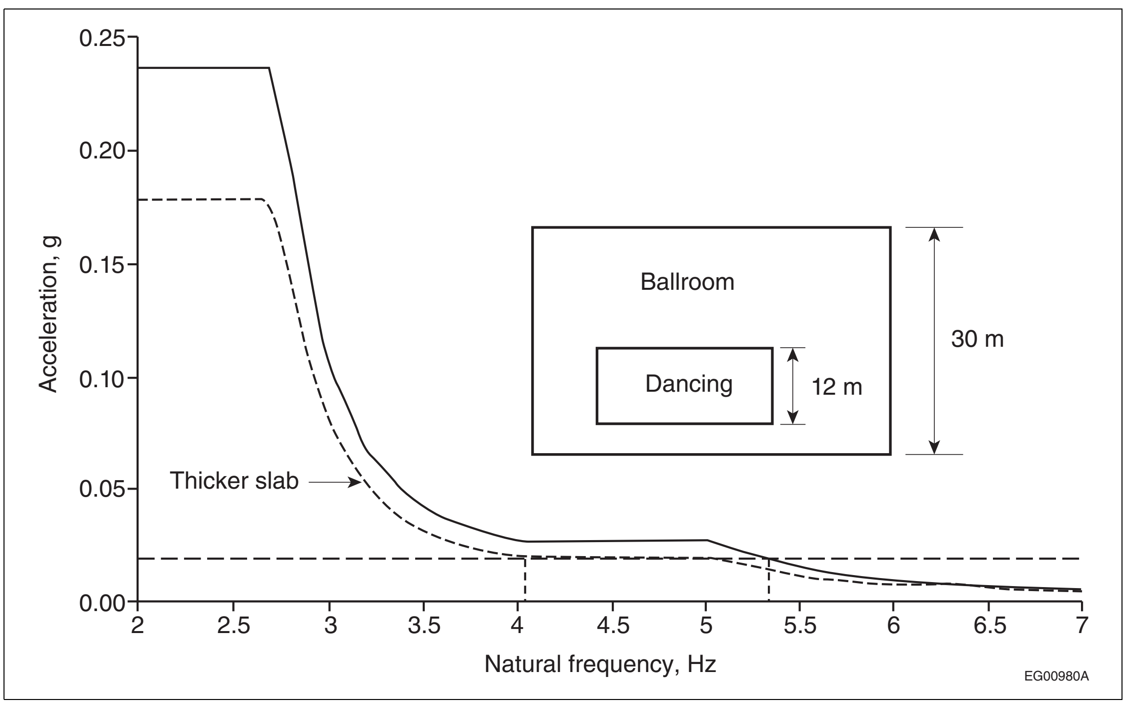

Figure D-4
Ballroom dining and dancing

17. A closer estimation of the minimum required floor frequency is obtained by applying Equations (4) and (5), where the effective weight of people is reduced from 0.6 kPa on the dance floor (from

Table D-2) to the equivalent of 0.24 kPa over the whole span (see floor layout in Figure D-4). The calculated maximum acceleration shown in Figure D-4 as a function of floor frequency is based on a damping coefficient of 0.06 and loading assumptions obtained from Table D-2. For a floor frequency of less than 2.7 Hz, vibration of 24% gravity acceleration occurs due to first harmonic resonance. For a floor frequency between 4 and 5 Hz, vibration of 2.7% gravity acceleration occurs due to second harmonic resonance.

18. To achieve an acceleration limit of 2% gravity, Figure D-4 shows that a floor frequency of 5.3 Hz is needed, which corresponds to a deflection of the floor structure, Δ, of 11.5 mm (span/2510): this is still not easy to achieve. However, Figure D-4 shows that an increased mass, w_t, resulting from the addition of 75 mm of concrete means a lower minimum frequency of 4 Hz is acceptable, which corresponds to a deflection of the floor structure of 20 mm (span/1500). The increased floor weight results in a moderate decrease of floor frequency. Alternatively, an FEM dynamic analysis to determine floor frequency might indicate that, without the extra concrete, a 5.3 Hz limit is achievable. More examples on the use of Equations (2) to (5) are contained in References [9], [10] and [16].

Measures to Prevent or Correct Unacceptable Vibration

19. Measures to prevent or correct unacceptable vibration due to rhythmic activities include:
- (a) applying administrative controls on rhythmic activities, such as by not allowing high-impact aerobics during office hours,
- (b) relocating the rhythmic activity or the sensitive occupancy,
- (c) providing sufficient stiffness (i.e., increased f_n) or mass (w_t) to satisfy the recommended criterion (see Equation (6)),
- (d) increasing the damping sufficiently to reduce resonant response, for example using tuned mass dampers,[9] or
- (e) providing isolation (floating floor) under jumping exercises to reduce dynamic forces at the second or third harmonic of the step frequency.[9]

For more guidance on correcting floor vibration, see References [9] and [10]. Case histories of problems are described in References [12] to [16], including a case where unacceptable aerobics vibration in a tall office building occurred due to the vertical spring action of the columns.

20. Based on recent documented experience with vibration problems in office and residential buildings, it is strongly recommended that, if an existing floor is intended to be used for aerobics or some other high-impact repetitive activity in the future, activity tests be carried out before making alterations or signing a lease. Such tests evaluate the performance of floors in nearby sensitive occupancies, including that of the floors above or below the activity.

Overloading

21. The total structural effect of a rhythmic activity can be determined from the static effect of the load, $w_t + \Sigma\varrho\alpha_i w_p$, where w_t is the total weight supported during the activity, and $\varrho\alpha_i w_p$ is the equivalent static load for the dynamic component at each harmonic, i, as defined in Paragraph 10. Overloading occurs if the total load, including static and dynamic components, is greater than the total specified load that the structure can support. A typical example is a floor structure with a floor frequency, f_n, of 5 Hz that supports aerobics. The most critical situation is second-harmonic resonance when high-impact aerobics is carried out at a step frequency of 2.5 Hz on a fully occupied floor. For the second harmonic, f equals f_n, and the dynamic amplification factor, ϱ, using Equation (1) equals $1/(2\beta)$. The equivalent static load, $\alpha_2 w_p/(2\beta)$, equals $0.6 \times 0.2/(2 \times 0.06) = 1$ kPa, where α_2 and w_p are obtained from Table D-2 and the damping ratio, β, is assumed to be 0.06. For the first harmonic, $f = 0.5f_n$ and $\varrho = 1.33$ using Equation (1). The equivalent static load, $\varrho\alpha_1 w_p$, equals $1.33 \times 1.5 \times 0.2 = 0.4$ kPa, where α_1 is obtained from Table D-2. The third harmonic with $\alpha_3 = 0.1$ and $f = 1.5f_n$ is very small. The dynamic component of load, $\Sigma\varrho\alpha_i w_p$, is therefore approximately equal to $1 + 0.4 = 1.4$ kPa, which is then rounded to 2 kPa to include all vibration frequencies. The total load, $w_t + 2$ kPa, is usually less than the specified dead load plus live load.

Fatigue

22. Potential for fatigue damage can be assessed by estimating the stress range and number of cycles per year for each harmonic. The stress range for each harmonic is equal to twice the stress due to the equivalent static load, $\alpha_i w_p \varrho_i$, or the static stress due to the load, $2\varrho\alpha_i w_p$. In the example given in Paragraph 21 of a 5-Hz floor used for aerobics, where second-harmonic resonance occurs, the stress range for the second harmonic is equal to the static stress due to 2 × 1 = 2 kPa, whereas for the first harmonic, it is equal to the static stress due to 2 × 0.4 = 0.8 kPa, while the third harmonic is very small. From field tests of aerobics vibrations, it is estimated that a typical session of high-impact aerobics lasts on average about 10 minutes, during 3 minutes of which second-harmonic resonance occurs. The first harmonic at 2.5 Hz is expected to have a duration of 10 minutes resulting in 1500 cycles, while the second harmonic at 5 Hz is expected to have a duration of 3 minutes resulting in 900 cycles. Other frequency components occur, but they are small and of short duration. Two sessions per day at 300 days per year result in approximately 1 million cycles per year for the first harmonic and 0.6 million per year for the second harmonic. The fatigue life for each harmonic can then be estimated from specified S–N fatigue curves in CSA S6, "Canadian Highway Bridge Design Code," and combined in accordance with the Palmgren-Minor rule.[10] In this example, a close estimate of fatigue life is obtained from the second harmonic only. However, design will almost always be governed by human reaction and not fatigue. For an existing building, it is recommended that fatigue life be based on vibration tests during a high-impact aerobics session to estimate acceleration levels versus number of cycles.

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Commentary E

Effects of Deformations in Building Components

Commentary E

Effects of Deformations in Building Components

Structural Effects

1. When building materials expand and contract due to temperature changes, considerable forces may be produced in restrained structural elements, i.e., those elements that are not free to expand and contract with the changes in temperature. Often these forces are compounded by those produced by shrinkage, creep and moisture content changes and are therefore difficult to analyze or predict. In many situations, however, the structural designer must consider the probable structural effects of the forces produced by temperature changes along with all other forces; indeed the designer is required to do so according to NBC Sentence 4.1.2.1.(1).

2. In addition to expansion and contraction, temperature changes may produce differential deformation or warping of materials as a result of a gradient in temperature through the thickness of materials or assemblies. Again this may complicate the assessment of deformations or stresses, but a rational judgment must be made in design if building elements are to perform in a satisfactory manner.

3. If these forces are not properly considered, the stresses resulting from such forces can lead to serious failures (usually cracking) in materials and structural members. Failures occur when clearances are insufficient, when fasteners do not allow movement or deformations, or, in the case of restrained elements, when the elements are not strong enough to withstand the stresses induced. An elementary review of thermal and moisture deformations in buildings is given in References [1] and [2]. Table E-1 indicates the order of magnitude of movement to which various materials are liable. Actual values can vary significantly from those in the Table.

Table E-1
Typical Deformation Properties of Some Common Building Materials

Material	Thermal Movement, mm/m per 100°C	Moisture Movement, mm/m		Modulus of Elasticity, MPa × 10^3	Creep Coefficient,[1] ϕ
		Permanent	Reversible		
Plain concrete[2]					
normal weight	1.0	0.5	±0.1	30	3
Glass	0.9	0	0	70	0
Masonry[3]					
clay	0.7	–0.2 (expansion)	±0.1	20	1
calcium silicate	1.0	0.2	±0.1	15	2
concrete					
normal weight	1.0	0.4	±0.2	15	2
lightweight (autoclaved)	1.0	0.4	±0.2	10	2
aerated (autoclaved cellular)	1.0	0.7	±0.2	6	2
Metal					
aluminum	2.4	0	0	70	0
copper	1.7	0	0	110	0
lead	3.0	0	0	14	0
steel	1.2	0	0	200[4]	0
Natural stone					
limestone	0.4	—	±0.1	60	0
marble	0.5	—	±0.1	35	0
sandstone	1.2	—	±0.3	20	0
Wood (spruce-pine-fir)					
across grain					
radial	4.0	30[5]	±Δmc[5]	1	[6]
tangential	6.0	50[5]	±2Δmc[5]	0.5	[6]
parallel to grain	0.4	1[5]	±Δmc/30[5]	10	1

(1) Deformation under sustained loading = short-term deformation based on modulus of elasticity × (1 + ϕ).
(2) For reinforced concrete, see CSA A23.3, "Design of Concrete Structures."
(3) For further information, see CSA S304, "Design of Masonry Structures."
(4) For cold-formed steel, see CSA S136, "North American Specification for the Design of Cold-Formed Steel Structural Members."
(5) Initial drying from green condition to equilibrium is assumed to be 12%; Δmc = per cent change in moisture content from 12%.[22]
(6) Such application is usually avoided.

Design Temperature Ranges

4. In a country like Canada, with its many climatic regions, the extremes of air temperature that have to be considered in the design of exteriors of buildings vary greatly. One way of approaching this problem is to use temperature maps like those given in CSA S6, "Canadian Highway Bridge Design Code," which give maximum summer and minimum winter air temperatures. Such a detailed approach may not be necessary for buildings. Instead, the 2.5% July and January air temperatures for the design of cooling and heating systems given in NBC Table C-2, Climatic Design Data for Selected Locations in Canada, in Appendix C of Division B are suggested. This will be illustrated by three examples below.

5. Because of solar heat gain in summer and radiation heat loss in winter, the range of temperatures that building elements are exposed to is greater than the ambient air temperature. Tables E-2 and E-3 show typical annual ranges of temperature differences between such elements and ambient air temperatures due to these effects.[2]

Table E-2
Temperature Increase in Excess of Ambient Air Temperature Due to Solar Radiation

Surface	Temperature Gain, °C
Dark roofing	20 – 40
Steel and other metal	15 – 25
Concrete and masonry	10 – 15

Table E-3
Temperature Decrease below Ambient Temperature Due to Radiation Loss into a Dark Clear Sky

Surface	Temperature Loss, °C
Dark roofing	10
Steel and other metal	5 – 10
Concrete and masonry	5

6. The values in Table E-2 vary according to the colour, slope, orientation and insulation backing of the surface.

Examples: For a horizontal dark-coloured metal surface in three typical climate regions (coastal, central and interior), the range of temperatures for design purposes might be as follows:

Coastal (Victoria):

$(24^{(i)} + 25^{(ii)}) - (-5^{(iii)} - 10^{(iv)}) = 64°C$

Central (Ottawa):

$(30^{(i)} + 25^{(ii)}) - (-25^{(iii)} - 10^{(iv)}) = 90°C$

Interior (Regina):

$(31^{(i)} + 25^{(ii)}) - (-34^{(iii)} - 10^{(iv)}) = 100°C$

7. Except for the very temperate parts of Canada referred to as Coastal, as a simple rule, one can assume a range of exterior surface temperatures of about 100°C for a horizontal relatively dark material. Because of thermal insulation, thermal inertia and other factors, however, the range of extreme temperatures in structural components of a certain thickness will often be somewhat smaller than those in the preceding examples.

8. Temperature variations can be particularly significant in multi-storey apartment and office buildings with exterior columns partially, and in some cases fully, exposed to the weather. Exposed columns, when subjected to seasonal temperature variations, change their length relative to interior columns, which remain unchanged in a controlled environment. Although this causes insignificant structural problems in low buildings, temperature stresses become significant in tall buildings and must be investigated thoroughly.

9. Dimensional changes occur not only as the result of temperature changes, but also from shrinkage, moisture content changes, chemical processes and creep deformation in the component materials of a building. If the building or component is not free to contract or expand, tensile or compressive stresses result. These stresses can be relieved or reduced to tolerable limits by contraction and expansion joints. Such joints are particularly important to allow contraction to take place along certain pre-selected lines rather than to produce cracks along accidental lines of least resistance.

(i) July 2.5% temperature.
(ii) Dark metal temperature gain.
(iii) January 2.5% temperature.
(iv) Dark metal temperature loss.

Effects on Cladding

10. In the design of all buildings, but particularly very long and very high buildings, the effects of movements of the structural members on the cladding elements should be considered. Shortening and lengthening of columns due to temperature and shrinkage effects and creep can crack, buckle or otherwise overstress cladding materials and their fastenings. Deflections and linear movements of beams and spandrels and building sidesway can have similar effects. Failure to consider these differential movements has caused many cases of cladding damage. For example, the brick and stone veneers on a number of tall concrete buildings have spalled, cracked and bulged,[3] necessitating extensive repairs. The phenomenon is not, however, limited to concrete frames, nor are the effects limited to stone and brick cladding. References [4] to [24] discuss these effects in greater detail.

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Commentary F

Live Loads

Commentary F

Live Loads

Notable Changes in this Commentary

- Introduction of commentary material addressing changes to the provisions in the NBC 2015 on guards, vehicle guardrails, elements within a guard, and walls acting as guards, as well as parking structures and repair garages
- Addition of commentary material on ultimate factored load combinations
- Addition of commentary material on office areas

Combined Live Load, L, and Snow Load, S (NBC Tables 4.1.3.2.-A and -B)

1. The load combination factors in NBC Tables 4.1.3.2.-A. and -B for ultimate limit states are based primarily on Turkstra's rule,[1] which is limited to the combining of statistically independent loads. Turkstra assumed that extreme values of different loads that vary in magnitude over time are unlikely to occur simultaneously. However, isolated cases of relatively high values of live and snow loads acting concurrently are possible; the potential for such situations should therefore be considered when designing the structure with load combinations for ultimate limit states.

2. In previous editions of the NBC, the factored combination of loads L and S resulted in very low values compared to the specified L and S when they were close in value. This discrepancy aroused concerns regarding building safety in areas with high specified snow loads. In 2008, many roofs in Quebec and Eastern Ontario were subjected to high snow loads sustained over a period of several weeks during which the use and occupancy of the floors below remained unchanged. This observation gave rise to the consideration that S should be addressed as being similar to a storage load in the context of its combination with L.

   As such, to account for S as a storage-type load, the companion-load factor for S has been set at 1.0 in the NBC 2015. The revised factored combination of L and S has minimal impact on the overall cost of building construction in general and has no impact at all on the cost of building construction in regions of Canada where the specified value of S is 1.5 kPa or lower and L is not considered a storage-type load. However, overall safety is significantly improved when the value of L is close to that of S in areas with moderate to heavy specified roof snow loads.

   If L is a storage-type load, the companion load factor for L is increased from 1.0 to 1.5 to account for the higher risk of its simultaneous occurrence with load S.

Considerations for Live Loads (NBC Table 4.1.5.3.)

Office Areas – Floors Above the First Storey

3. NBC Table 4.1.5.3. provides a 2.4 kPa specified live load as a minimum requirement for office areas above the first storey not including special designated areas for record storage, libraries, or computer rooms. Typical modern office buildings often include these special designated areas for which the minimum specified live load of 2.4 kPa may not be adequate and the floor use may be storage in nature. In addition to the need to design the floor structure for the live loads for these designated areas within an office building, the tributary area reduction factor in NBC Sentence 4.1.5.8.(2) will

likely apply. The location, associated live load, and the type of live load for these special designated areas should be noted on the structural drawings.

A common record storage device used in office buildings is the high density mobile shelving system, a compact assembly of filing cabinets that are supported on rails mounted on to the office floor. The ability of the structural system to accommodate these systems without modification to the structure is highly dependent on the structural system. For example, a thick concrete structure might more easily accommodate the future installation of these systems than a thin concrete slab on steel deck. This system is popular for office space owned or leased by government agencies and corporations with head office facilities. It should be noted that the use of mobile shelving systems and other special designated floor uses may not be identified at the time of the design of the office building or may be added many years after the original occupancy of the building. There is evidence that some owners and lessors of office buildings are mandating that the specified office floor live load be greater than the minimum NBC requirement of 2.4 kPa in recognition of the need for flexibility in the installation of high density mobile shelving systems which may not be identified at the time of the design and construction. Designers should be aware of these circumstances and seek specific instruction from the owner as to what accommodation should be made.

Office Areas – Basement and the First Storey

4. NBC Table 4.1.5.3. requires the minimum specified live load for the basement and first storey to be 4.8 kPa; the intent is to apply the 4.8 kPa loading to all office floors with direct access to exterior ground.

Sidewalks and Driveways Over Areaways and Basements

5. The minimum specified live load for sidewalks and driveways over areaways and basements is a uniform load of 12 kPa or a single concentrated load of 54 kN as stated in NBC Article 4.1.5.9. NBC Note A-Table 4.1.5.3. provides guidance on additional loading cases for vehicles exceeding 9 000 kg gross weight.

Concentrated Loads (NBC Article 4.1.5.9.)

6. The size of the loaded area for concentrated roof and garage floor loads was redefined in the NBC 2010 to account for actual situations. The concentrated load for roof area is based on the footprint of a workman with tools while the concentrated load for garage area is based on a vehicle jack baseplate area during vehicle maintenance.

Tributary Area

7. Because live loads are generally given as uniformly distributed loads over a floor area, and because dead loads can usually be considered as uniform loads, either over an area or along the length of a flexural member, design engineers have for years used the concept of tributary area to determine the loads that beams, girders and columns carry. Once the concept is applied to any floor, it is easily extended for multi-storey columns to any number of floors.

8. Earlier design standards recognized that the probability that all the floors of a multi-storey building would be loaded to the full live load simultaneously was very remote. Therefore, to design the columns for the full live load of a number of floors was unduly restrictive, and reductions in the live load were devised as a function of the number of floors supported by the columns.

9. In the 1960 edition of the NBC, recognizing that the average live load was a function of the area supported, the rationalization was carried one step further and a reduction of 15% was allowed for beams, girders and trusses supporting areas greater than 20 m^2.

10. In subsequent editions, provisions were included for live load reduction based on tributary areas with two different expressions—one for office and apartment buildings and the other for storage and similar areas.

11. Therefore, for determining the total dead load to be supported by a given member and to determine what live load reduction factor should be applied, a clear definition of tributary area, about which some confusion existed, is needed.

12. In the case of a member that supports the load directly, such as a slab, the tributary area is defined as the area supported by the member bounded by the lines of support. In the case of a member that does not support the load directly but supports other members, the tributary area is defined as the area bounded by the lines of support of the member and the lines of zero shear in the members supported, assuming a uniformly distributed load is acting on the structure. These definitions should be followed when determining the forces that members carry (continuous construction would require a structural analysis to determine the locations of zero shear). In determining live load reduction, however, the following simplifications are recommended.

Decks and Slabs

13. No live load reduction factors should be applied to wooden or sheet metal decks, precast units or one-way slabs because of the uncertainty of the degree of lateral distribution of loads.

14. The tributary area for a flat slab or the slab portions of two-way slabs with beams is the area bounded by column lines or by a combination of column lines and lines of supporting members such as beams and girders, whichever is the lesser, as shown in Figures F-1, F-2 and F-3.

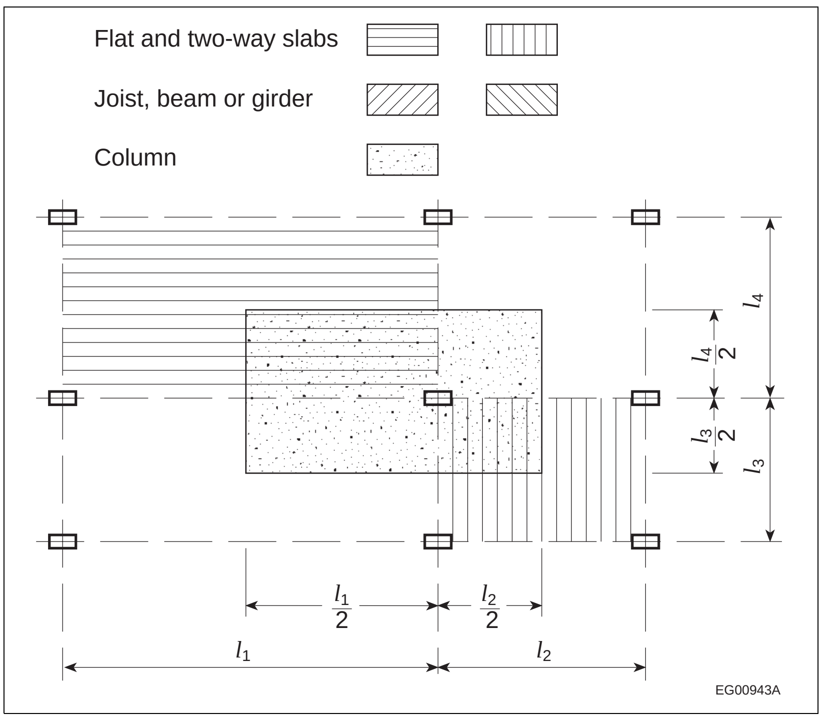

Figure F-1
Tributary areas for flat slabs without beams and girders

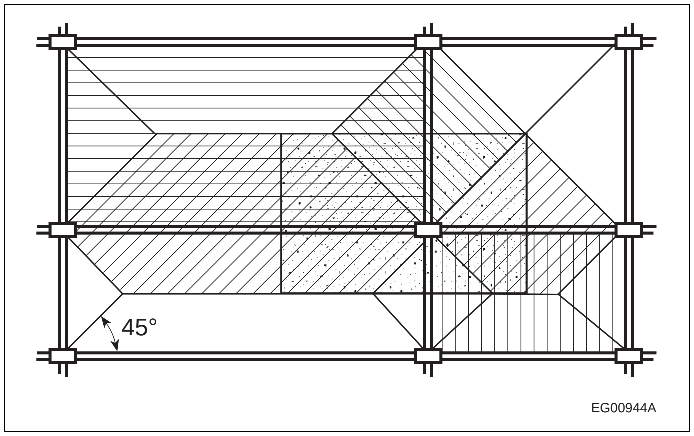

Figure F-2
Tributary areas for a two-way slab with beams

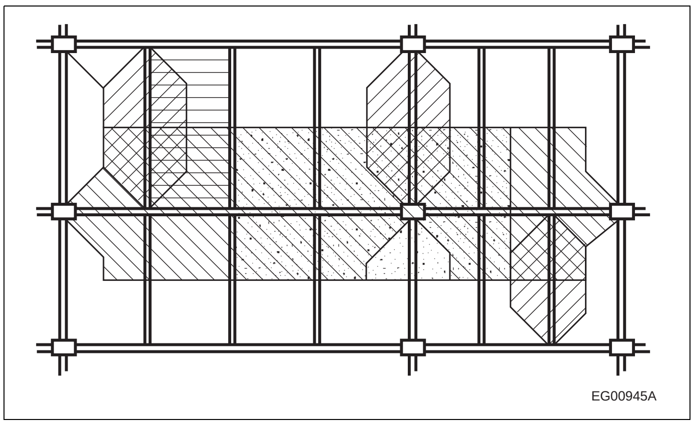

Figure F-3
Tributary areas for a two-way slab with joists, beams and girders

Beams and Girders

15. The tributary area for a member supporting a portion of a floor is the area enclosing the member and bounded by the lines of zero shear in the members supported. For buildings with fairly regular bays, the lines of zero shear in the members supported can be assumed to be halfway between lines of support. Figures F-2 and F-3 illustrate the tributary area of beams supporting two-way slabs. Figures F-4 and F-5 illustrate the tributary areas for joists, beams and girders supporting a one-way slab.

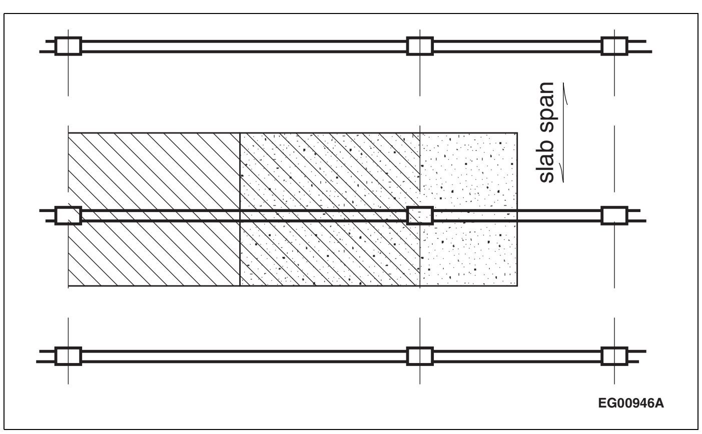

Figure F-4
Tributary areas for a one-way slab with girders

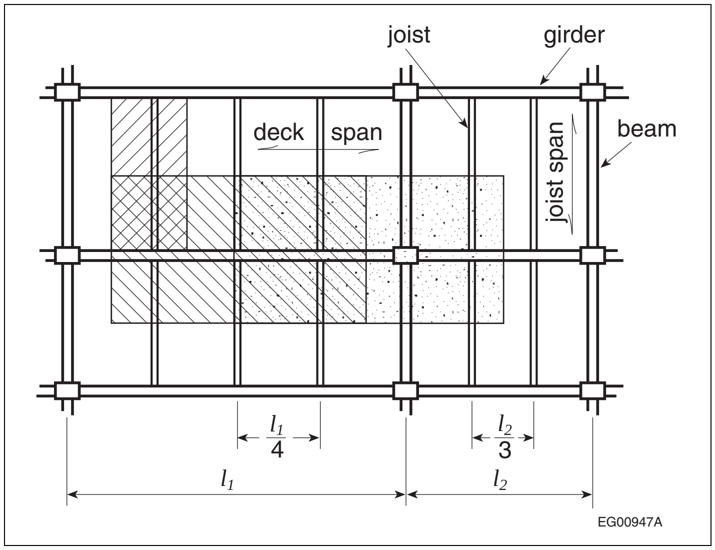

Figure F-5
Tributary areas for a one-way deck or slab with joists, beams and girders

Negative Moments in Continuous Members

16. Tributary area for negative moment over a support may be taken as the sum of the tributary areas of the beams on either side of the support. For cantilever, drop-in beam systems, the tributary area for calculation of the live load reduction factor, as it affects the negative moment at the support, is taken as the sum of the areas tributary to the cantilevered section and one half the length of the drop-in section.

Columns

17. For a column, the tributary area per floor is the area of floor supported, bounded by the lines of zero shear. For buildings with fairly regular bays, these can be assumed to be halfway between the column lines, as shown by the dotted area in Figures F-1 to F-5. In structures with beams, joists or girders, the tributary area per floor is half the sum of the tributary areas of each of the floor members framing into it.

18. In multi-storey buildings, the tributary area for a column supporting one use and occupancy is the sum of the tributary areas per floor for that column on all levels above the storey in question.

19. For a column supporting more than one use and occupancy, NBC Article 4.1.5.8. requires that the tributary area for each use and occupancy be considered separately for determining reduction in live load and that the area supporting snow load, which has no reduction, not be included.

Loads on Guards (NBC Article 4.1.5.14.)

Applied Loads on Guards

20. The minimum specified horizontal loads applied inward on guards have been decreased to 50% of those applied outward in recognition of the fact that the consequence of malfunction of a guard upon inward application of a load is not as severe as a malfunction upon outward application of a load (i.e., a person falling inward towards the floor or building or other supporting element versus them falling into empty space). This change also more closely reflects real-life loads on guards.

Lateral Loads on Elements within a Guard

21. NBC Sentence 4.1.5.14.(3) specifies that individual vertical elements within guards must be able to withstand a load of 0.5 kN applied outward and perpendicular to the guard. Because of the magnitude of the load, these elements must be designed to have sufficient stiffness in the direction of the load to comply with the NBC and to limit deflections to acceptable levels as required by CSA material standards.

22. In recent years, more and more guard designs have incorporated thin vertical aesthetic elements that exhibit large deflections when subjected to relatively small loads in the plane of the guard system; i.e., people could easily pull them apart, thereby compromising the maximum clear distance between adjacent vertical elements specified in NBC Part 3. New NBC Sentence 4.1.5.14.(4) limits the size of the opening between adjacent vertical elements within a guard to that required in NBC Part 3 when they are subjected to a specified load of 0.1 kN applied in opposite directions in the in-plane direction of the guard. The location of application of the load that is likely to produce the most critical effect is at mid-height of the vertical elements. Inspectors can easily approximate the 0.1 kN load by manually pulling apart the elements under scrutiny.

Deflection of Guards

23. Whereas NBC Article 4.1.5.14. clearly states the minimum specified loads that guards must be designed to withstand, maximum acceptable deflection is not directly addressed in the Code. General performance requirements are given in NBC Article 1.2.2.1. of Division A, which calls for “all materials, appliances, systems and equipment... to possess the necessary characteristics to perform their intended functions,” and in NBC Clause 4.1.3.5.(1)(a), which seeks to limit serviceability problems due to deflections. Although no specific maximum deflection limits are expressly stated, it is understood that guards—in their entirety—must be sufficiently rigid to meet these requirements and resist the potential disconnection of elements within them.

In calculating the deflection of a guard, the live load is applied at the minimum required height of the guard as required by NBC Part 3 and the wind load is applied over the surface area of the guard. The resultant maximum deflection under the action of these loads applied using the serviceability load combination factors should not exceed any of the deflection limits stated in ASTM E 985, “Permanent Metal Railing Systems and Rails for Buildings.”

Guards should be designed to resist permanent deflection and not malfunction from fatigue due to repetitive deflections under normal service loads.

Design Loads for Guards for Ultimate Limit States

24. Guards must be designed to withstand not only the specified loads stated in NBC Articles 4.1.5.14. to 4.1.5.16. but also the companion loads addressed in NBC Article 4.1.3.2. using the load combinations presented in Table 4.1.3.2.-A.

Vehicle Guardrails (NBC Article 4.1.5.15.)

25. The outward load application referred to in NBC Sentence 4.1.5.15.(1) is assumed to be in the direction of travel of vehicles and perpendicular to the guardrail.

Where vehicle guardrails are installed to protect pedestrians from vehicle impact and not to prevent vehicles from falling from one level to another in a parking structure or roadway, for example, they need only be designed to withstand the loads specified in NBC Article 4.1.5.15. and need not serve as a guard in accordance with the definition given in NBC Article 1.4.1.2. of Division A.

Where a guard in the sense of the definition is also required at the same location as a vehicle guardrail, either one element can be installed that serves as both a vehicle guardrail and as a guard, or a guard can be erected that is independent of the vehicle guardrail.

Where an element serves as both a vehicle guardrail and a guard, it must satisfy the requirements for guards stated in NBC Parts 3 and 4 and those for guardrails stated in NBC Part 4. However, the requirements for each type of guard need not be assumed to be applied simultaneously.

Loads on Walls Acting As Guards (NBC Article 4.1.5.16.)

26. The live load referred to in NBC Article 4.1.5.16. that produces the most critical effect is to be considered independently as a principal or companion live load for all load combinations listed in NBC Tables 4.1.3.2.-A or -B, as applicable. There are many configurations of interior and exterior walls adjacent to a drop in elevation of more than 600 mm that would need to be checked for all such possible load combinations, including where the wall is full-height between floors, partial-height, cantilevered upwards from a floor, or a spandrel wall that projects partially above and below a floor.

Example in Figure F-6: In each load case shown, the wall would have to be analyzed for all lateral load combination cases using both L and W as follows:

Load Case from NBC Table 4.1.3.2.-A	Principal Load	+	Companion Load
2	1.5L	+	0.4W
4	1.4W	+	0.5L
5	1.0E	+	0.5L

where
E = earthquake load,
L = applicable guard load applied at appropriate height for load case A or 0.5 kPa for load case B, and
W = worst-case outward uniform wind load on the wall consisting of the summation of internal positive pressure and external negative pressure.

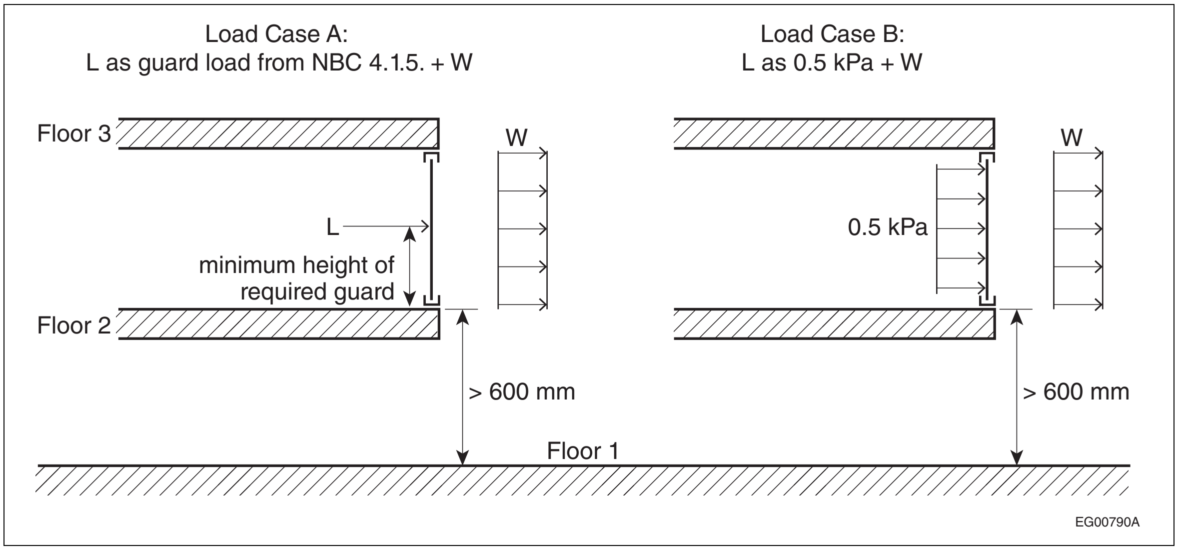

Figure F-6
Load cases on a full-height wall adjacent to a drop in elevation of more than 600 mm

Design Basis for Parking Structures and Repair Garages (NBC Sentence 4.4.2.1.(1))

27. CSA S413, “Parking Structures,” applies to all parking structures or parts thereof constructed of structural steel, reinforced concrete, pre-stressed concrete, or any combination of these materials. Such structures are subject to accelerated deterioration through the action of de-icing chemicals and other environmental factors, such as water, which are tracked in from the outside by vehicles or pedestrians or are intentionally applied to concrete surfaces.

The intent of CSA S413 is to increase the longevity of these structures and to prevent the leakage of water through the slabs on one level to the surface of the level below.

Some parts of building structures, such as balconies, stairs, ramps, pedestrian walkways and exterior entrances to buildings, are also subject to accelerated deterioration for the same reasons that parking structures are. Corrosion protection should be a consideration in the design of these areas.

Unprotected reinforced concrete slabs subject to de-icing chemicals or other environmental factors can begin deteriorating early in their service life, thus requiring significant repair. However, buildings constructed of such slabs that are designed and maintained in accordance with the minimum requirements of CSA S413 are expected to last much longer.

The basis of the corrosion protection methods presented in CSA S413 is Life-365™, a software model that predicts the service life and life-cycle cost of reinforced concrete exposed to chlorides.

Designers and developers should carefully assess the service life expectancy of building elements subject to chloride attack and the costs associated with maintaining, repairing and replacing such elements.

28. Where materials other than steel and concrete are used to construct a parking structure, designers must ensure that its performance will meet the objectives of NBC Part 4 under the conditions of the parking structure's use and location.

29. Repair garages are also subject to accelerated deterioration for the same reasons that parking structures are. Accordingly, CSA S413 also applies to the design of repair garages.

Hand- or machine-troweled finishes may need to be applied to floors of repair garages due to their function as a vehicle repair facility. It is important to note that the application of a hardener in the finishing of a slab of air-entrained concrete will likely result in blistering of the hardener and/or debonding of the concrete as it cures. However, toppings with hardener qualities and thick-set membrane toppings can be successfully applied to concrete slabs with air entrainment if they are bonded to the top surface of the slab after the concrete has sufficiently cured. Designers should verify the degree of curing of concrete that is required and the compatibility of the selected topping with air-entrained concrete.

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Commentary G

Snow Loads

Commentary G

Snow Loads

Notable Changes in the National Building Code of Canada (NBC) 2015

- Relocation of essential guidance on the determination of snow loads from this Commentary to the NBC with a view to centralizing the information designers need to carry out their work
- Modification of the calculation of the basic roof snow load factor, C_b, based on new research and tabulation of the values of C_b for various roof sizes and wind exposure factors
- Introduction of a method for calculating the specific weight of snow in drifts
- Relocation (from this Commentary to the NBC) and modification of the calculation of the accumulation factor, C_a, (previously called the shape factor)
  (1) modification of the calculation of C_a for multi-level roofs and roof projections to better account for drifting from an upper roof into a roof step, for drifting across a lower roof into a roof step, and for drifts against a roof projection, using a more consistent methodology to cover all three types of accumulation
  (2) reduction in number of load cases in the calculation of C_a for arch and curved roofs from three to two
  (3) modification of the calculation of C_a for domes
- Relocation of the calculation of snow loads due to sliding from an upper roof onto a lower roof from this Commentary to the NBC and modification of the calculation to ensure that the extra load on lower roofs is considered for all slopes of upper roofs when they are slippery
- Relocation of the calculation of snow loads in valleys of curved or sloped roofs from this Commentary to the NBC
- Introduction of a prohibition on the reduction of design snow loads on the basis of snow removal by various means
- Introduction of a provision concerning ice loading on lattice structures or other building components or appurtenances that references the CSA S37 standard

Notable Changes in this Commentary

Discussions on the following subjects have been added:

- arch roofs and domes with geometries not covered in the NBC
- multiple-gable roofs with non-parallel roof lines
- calculations of drift loads
- snow guards
- effect of solar panels on snow loads

Snow Loads on the Ground

1. In Canada, ground snow loads are used as a basis for the determination of roof snow loads. Therefore, they form part of the basic climatic information needed for building design and are presented in NBC Table C-2, Climatic Design Data for Selected Locations in Canada, in Appendix C of Division B. Ground snow loads consist of two loads: S_s, which is a snow load with a 1-in-50 annual probability of exceedance based on measured depths and densities, and S_r, which is due to the associated rain that may fall into the snow cover (not including any rainfall that exceeds the weight of the snow cover).[1] (See Paragraph 4.)

The snow loads for a given town or city are for the exact latitude and longitude defined in the Canadian Geographical Names Data Base (CGNDB)[2] for that town or city. Snow loads may vary within cities with large changes in elevation. Recommended values of S_s and S_r for significantly different elevations within listed sites and for locations not listed in NBC Table C-2 can be obtained from the Meteorological Service of Canada of Environment and Climate Change Canada; e-mail: ec.enviroinfo.ec@canada.ca. Elevations are not given in the CGNDB but can be obtained from commercially available topographic maps prepared by Natural Resources Canada.

Variations with Climate

2. The wide climatic variations across the country produce large variations in snow conditions. The heaviest snow loads occur in the mountainous regions of British Columbia and Alberta; they last the entire winter and vary considerably with elevation. In some coastal locations of British Columbia, little drifting of snow occurs. The Prairie provinces, Yukon, Nunavut and the Northwest Territories have very cold winters, with small annual snowfalls but frequent strong winds, which cause considerable drifting of snow on roofs and on the ground. The region that includes Ontario, Quebec, and interior regions of the Atlantic provinces is marked by moderate winds and snowfalls, and sufficiently low temperatures in most places to allow snow accumulation all winter. In this region, moderate uniform and high drift loads occur. Also, cold northwesterly winds often cause locally heavy snowfalls to the lee of bodies of water such as the Great Lakes and the St. Lawrence River, resulting in increased snow loads.

Local Variations – Mountainous Areas

3. In mountainous areas, ground snow loads increase with elevation. Observations noted by the Institute for Research in Construction of the National Research Council of Canada (now called NRC Construction) on a number of mountains in British Columbia indicate significant increases in ground snow load with increases in elevation, depending on the local topography and climate.[3] Individual mountains or groups of mountains may cause significant changes in a local or micro climate within short distances. Hence, snow loads listed in NBC Table C-2 apply only at a particular elevation at the specific location as defined by the name and latitude/longitude coordinates given by the CGNDB.[2] Environment and Climate Change Canada should be consulted for specific recommendations regarding other significantly different elevations within a listed location. (See also Paragraph 1.)

Specific Weight of Snow on the Ground

4. Falling snowflakes usually consist of very large complex ice crystals. Because of their large ratio of surface area to weight, they fall to the ground relatively slowly. On arrival, this snow accumulates in a loose and fluffy layer with a specific weight, γ, of about 0.5 to 1.0 kN/m^3. Immediately, however, the snow crystals start to change: the thin, lacy, needlelike projections begin to sublime and the crystals become smaller, irregularly shaped grains. Settlement of the snow results and the specific weight, γ, increases after a short time to about 2.0 kN/m^3 or greater, even at temperatures below the freezing point. The specific weight of the snowpack continues to increase with age, ranging from 2.0 to 5.0 kN/m^3. As explained in NBC Appendix C of Division B, average values for seasonal snowpacks have been derived for different regions across the country for use in the ground snow load calculations.[1] The snow surveys from which γ is derived are made up to four times per month. While the survey measurements reflect to some extent the portion of rainfall that is trapped in the snowpack over a period of time, only a small proportion of measurements would have been made directly after a rainfall. Therefore, the measurements probably do not adequately represent the short-term specific weight increase due to the wetting of snow by rain; for this reason, the rain load, S_r, is included in the calculation of roof snow loads.[1]

Snow Loads on Roofs

5. Snow loads on roofs vary according to geographical location (climate), site exposure, shape and type of roof, and also from one winter to another. To account for these varying conditions, NBC Subsection 4.1.6. expresses the specified snow load, S, on a roof or other surface as the sum of two components—one being the product of a series of factors—multiplied by an importance factor:

$$S = I_s[S_s\,(C_b C_w C_s C_a) + S_r] \qquad (1)$$

where

I_s = importance factor for snow load,
S_s = ground snow load, in kPa, with a 1-in-50 probability of exceedance per year,
C_b = basic roof snow load factor,
C_w = wind exposure factor,
C_s = roof slope factor,
C_a = accumulation factor, and
S_r = associated rain load, in kPa, (however, the rain load at any location on a roof need not be taken greater than the load due to snow, i.e., $S_r \leq S_s(C_bC_wC_sC_a)$).

These factors are based on measurements obtained during surveys of snow on roofs,[4] on analytical studies of the loads on large flat roofs, on experience, including failures, and on judgment; they are discussed individually in Paragraphs 10 to 21.

Specific Weight of Snow on Roofs

6. To calculate loads due to snow on roofs, an estimate of the specific weight of snow, γ, is necessary. Measurements taken at a number of weather stations across Canada resulted in values of γ that ranged from about 1.0 to 4.5 kN/m^3. Working values of γ suggested in previous editions of this Commentary ranged from 2.4 kN/m^3 to 3.0 kN/m^3.[5] However, the specific weight of snow can be even higher in some locations such as regions where the maximum roof load is reached only after contributions from many snowstorms, coastal regions, and regions where winter rains are considerable; in such locations, a value of γ as high as 4.0 kN/m^3 may be appropriate. The specific weight of snow tends to increase with higher snow loads and roof snow loads tend to increase where the ground snow load is higher. ASCE/SEI 7, "Minimum Design Loads for Buildings and Other Structures," contains a formula to calculate the increase in the value of γ based on an increase in the ground snow load: $0.43\ S_S + 2.2$ kN/m^3. This formula provides results that are reasonably consistent with Canada's climatic reality and so it has been added to NBC Article 4.1.6.13. so that designers have a common basis for determining the specific weight of snow on roofs. The specific weight of snow is capped at 4.0 kN/m^3 as higher values are extremely rare.

Solar Radiation and Heat Loss

7. Some factors that modify snow loads occur only under special conditions. For example, solar radiation has little effect in reducing loads in cold weather. Similarly, during cold weather, heat loss from roofs is not very effective in melting the snow, particularly on well insulated and well ventilated roofs. These two factors cannot, therefore, be relied upon to significantly reduce the snow load during colder periods. During thaws and toward the end of winter, however, when air temperatures approach the freezing point, solar radiation and heat loss do cause melting.

Roof Snow Load Factors

8. The factors C_b, C_w, C_s and C_a were obtained through research and field observation because rigorous statistical analyses have, in most cases, not been possible due to the lack of data. These factors have nonetheless been found to provide roof designs that perform acceptably in practice.

9. **Basic roof snow load factor, C_b.** The basic roof snow load is set at 80% of the ground load (i.e., $C_b = 0.8$), except for larger roofs. This percentage is based on the results of a countrywide survey of snow loads on roofs carried out by the Institute for Research in Construction of the National Research Council of Canada (now called NRC Construction) and a number of volunteers. The wind is less effective in removing snow from large roofs due to the greater quantities involved and because snow may drift from one area to another.[6] Increased values of C_b are therefore specified in NBC Clauses 4.1.6.2.(2)(a) and (b) to account for this effect in the case of large roofs.

   Note that when $C_w = 1$, C_b may be calculated as follows:

$$C_b = 0.8 \text{ for } l_c \leq 70\text{ m, and}$$
$$C_b = 1 - 0.2\exp\left(-\frac{l_c - 70}{100}\right) \text{ for } l_c > 70\text{ m}$$

   where l_c is the characteristic length of the upper or lower roof, in m, as per NBC Sentence 4.1.6.2.(2).

10. **Wind exposure factor, C_w.** Observations in many areas of Canada have shown that where a roof or a part thereof is fully exposed to wind, some of the snow is blown off or prevented from accumulating, thus reducing the average snow load.

11. For roofs fully exposed to the wind, the wind exposure factor, C_w, may be taken as equal to 0.75 in rural areas only rather than 1.0 (or 0.5 rather than 1.0 for exposed sites north of the treeline). This substitution applies under the following conditions:
   (a) the building is on open level terrain containing only scattered buildings, trees or other such obstructions, open water or shorelines thereof, and is expected to remain so during its service life;
   (b) the area of roof under consideration is exposed to the wind on all sides and does not have any significant obstructions, such as parapet walls, within a distance of at least 10 times the difference between the height of the obstruction and $C_bC_wS_s/\gamma$ metres, where the applicable value of C_w is either 0.75 or 0.5, as provided in NBC Sentence 4.1.6.2.(4);
   (c) the loading case under consideration does not involve the accumulation of snow due to drifting from adjacent surfaces such as, for example, the other side of a gable roof; and
   (d) the buildings are not in the High or Post-disaster Importance Categories described in NBC Table 4.1.2.1.

   A value of 1.0 for C_w must be applied to snow loads that involve drifting from adjacent surfaces.

12. The value of $C_bC_wS_s/\gamma$ is the height of uniformly distributed snow on a roof without any obstructions, including parapets. Any obstructions lower than this do not generate additional snow loading.

13. In practice it is sometimes difficult to make a clear distinction between roofs that will be fully exposed to winds and those that will not. The designer should, in consultation with the owner, weigh the probability of the roof becoming sheltered by an addition to the building or by adjacent higher buildings or trees. Such changes could cause either drift loads or higher average loads. In considering drift loads—which are the more serious—a minimum distance of at least 5 m should be maintained from another existing or future building or from the property line to justify disregarding drift loads. This corresponds to the distance used in NBC Clause 4.1.6.2.(8)(a) for multi-level roofs. With regard to higher average loads, it is important to use a wind exposure factor, C_w, equal to 1.0 for any roof area whose exposure may decrease. For that matter, given the uncertainties regarding future exposure, C_w should be 1.0 in practice in most cases.

14. The designer should also be aware that the snow loads on the roof of an existing building on the same or adjacent property may also be affected by the location of a new higher building or other obstruction.

15. The installation of solar collectors on roofs may result in higher snow loads similar to those around obstructions, unless the clear gap under them is sufficiently large to allow scouring and removal by the wind rather than deposition.[7]

16. **Roof slope factor, C_s.** Snow loads on a sloping surface act on the horizontal projection of the surface. Under most conditions, less snow accumulates on steep roofs than on flat and moderately sloped roofs, because of sliding, creep, better drainage and saltation.[8][9][10] The coefficient, C_s, as defined in NBC Sentence 4.1.6.2.(5), accounts for these effects by reducing the snow load linearly from full snow load at 30° slope to zero at 70°. A lesser value of C_s is permitted in NBC Sentence 4.1.6.2.(6) for unobstructed, smooth, slippery roofs, such as those made of glass or metal. In this case, the load may be reduced linearly from full load at 15° to zero at 60°. In order for the designer to use the full reductions as described in either of these relationships, the snow should be able to slide completely off the roof surface under consideration.

17. Situations in which public safety may be compromised by snow and ice falling from roofs should be avoided. If snow fences or barriers are required to keep snow and ice on roofs, they should be designed to transmit the substantial forces involved into the building structure.[8][9] Heat-traced gutters, heated drips or some other means to prevent the growth of dangerous icicles due to meltwater from the snow retained on roofs may also be required. Snow and ice falling from the roof of a building may be deflected against the building and cause damage.

18. **Accumulation factor, C_a.** The accumulation factor, C_a, for a number of different roof shapes is addressed in NBC Articles 4.1.6.5. to 4.1.6.12.; C_a for all other roof shapes should be determined by the designer based on applicable field observations, special analyses usually accounting for

local climate effects,[11][12][13] on model tests,[7] or a combination of these methods. In an effort to provide guidance, the Institute for Research in Construction of the National Research Council of Canada (now called NRC Construction) published two collections of informative case histories[14][15] on non-uniform snow loads.

19. **Drift accumulation on roofs.** When the wind encounters obstructions, regions of accelerated and retarded flow result. The regions of retarded flow are said to be regions of "aerodynamic shade."[16] Because a minimum velocity is required to transport the snow, it settles out where the flow velocity is too low and forms drifts whose shapes are indicated by C_a. In general, the longer the wind duration, the deeper the drifts on roofs, especially if it is also snowing; additionally, the greater the wind speed, the less uniform the snowdrifts.

20. Roofs situated below adjacent roofs are particularly susceptible to heavy drift loads because the upper roofs can contribute a large volume of snow that gathers in drifts[5][6][17][18][19] and snow drifting across the lower roof can also gather in drifts at the bottom of the step change in roof elevation. These types of accumulation are addressed in NBC Article 4.1.6.5. by load Cases I and II. Canopies, balconies and porches are similarly susceptible. The drifts that accumulate on these roofs and platforms depend mainly on the difference in elevation and on the size of the upper and lower roofs.[6]

21. Where the lower level roof area is large, wind blowing for a considerable time at an angle towards the raised portion of the roof may form an elongated "spike" or quartering drift extending leeward of the change in elevation.[20] This type of accumulation is addressed in NBC Article 4.1.6.5. by load Case III.

22. NBC Article 4.1.6.5. covers the typical types of snow load that arise on lower level roofs. For unusual geometries, especially where the roof areas involved are large, model studies can be useful in identifying unusual drift formations.

23. Projections such as penthouses or parapet walls on flat roofs may collect trapezoidal snow drifts but the magnitude of the loads is usually less than that of snow loads on roofs situated below adjacent roofs. NBC Article 4.1.6.7. addresses the calculation of this type of loading.

24. Wind flow accelerates over gable and arch roofs because it is deflected upwards on the windward sides. On the leeward sides, velocities drop and the snow entrained in the wind and scoured from the other side is deposited. Heavy unbalanced loads often occur as a result of the transfer of snow from one side to the other.[21][22] This unbalance is especially important for domes and for buildings such as arenas, which have long spans and in which a collapse might be catastrophic.[4] Lightweight curved structures, such as cold-formed metal arch buildings, are particularly sensitive to unbalanced snow loads as the self-weight of the structure is relatively small. These structures can generally be analyzed as arches. However, the flexibility of such arches suggests that a second-order analysis is likely to be required to predict their structural behaviour.[17][21] The structures can also be analyzed as shells when special consideration is given to shear transfer and to the axial capacity of longitudinal stiffeners. Load tests may be needed to assess the behaviour and load-carrying capacity of the structural elements, especially when transverse corrugations are present. NBC Articles 4.1.6.9. and 4.1.6.10. present balanced and unbalanced load cases for gable, arch and curved roofs as well as domes.

25. Wind tunnel and water flume tests, such as those described in ASCE/SEI 49, "Wind Tunnel Testing for Buildings and Other Structures," are recommended to assist in the selection of appropriate design loads for arch roofs and domes with geometries not covered by NBC Article 4.1.6.10.

Local experience should also be considered. Snow accumulations due to sliding and drifting occur regularly at the bases of domes where they meet the ground; these should not be overlooked.

26. In windless areas, snow covers roofs and the ground in uniform layers. For these locations, the design load can be considered as a uniformly distributed load equal to some suitable fraction of the ground snow load if sliding is not a factor. Truly uniform loads, however, are rare and have been observed only in certain mountain valleys of British Columbia (BC) and occasionally in other parts of the country, on roofs that are well sheltered on all sides by high trees. Generally, the winds that usually accompany or follow snowfalls transport new snow from exposed to protected areas. Hence, the probability that high uniform loads will occur on exposed roofs is reduced and the probability that drifts will form is increased. Drifting does not occur in certain areas on the BC coast where heavy snowstorms invariably consist of wet snow. In such locations, the drift requirements of NBC

Sentence 4.1.6.2.(8) may be overly conservative. Where the authority having jurisdiction is convinced that drifting will not occur, drift effects need not be considered. However, the influence of creep and sliding snow causing unbalanced loads should be considered for gable roofs with a slope > 15°, arches with a height-to-span ratio, h/b, greater than 0.05, and other roofs with a significant slope.

27. In high snowfall areas of Canada, in particular the mountains and valleys of British Columbia, the following should be kept in mind:[5][23]

(a) Snow cornices can become very large and cantilever beyond the edges of flat or sloping roofs a distance equal to the depth of the snow on the roof. This can occur in sheltered or windless areas and on the leeward side of roofs where there is wind. Cornices have been known to overload walls and columns, resulting in failures. In addition, cornices are a hazard if they break off. They can destroy balconies, stairs, porches, attachments of wires, etc., to the building and can be very dangerous to people below.

(b) When a large amount of snow is deposited on slippery sloping roofs, it has been known to shear off vents, chimneys, aerials, wiring, stacks, skylights and ventilators when it slides. It is a menace to people and things below when it falls. In addition, it may creep off the roof, rotating slowly at the eaves, and may even break windows if it hits the side of the building. Protrusions through the roof should be located at the ridge or be especially protected against the shearing forces of sliding snow.

(c) Where a roof is L-shaped or has dormers, the snow on each slope will slide in the direction of the ribs or corrugations and accumulate in the valleys. If one slope is longer or steeper, the snow on this slope will predominate and may force the whole mass of snow to slide across the opposing corrugations on the other slope, resulting in a tearing or flattening of these corrugations. If the corrugations hold and the snow does not slide, the restraining load on the lower opposing slope may be very high.

28. **Redistribution of load due to melting.** Loads may get redistributed on roofs as a result of snow or ice melting and flowing or sliding to other areas where it refreezes, or falling to a lower roof where it accumulates as slush or ice. Meltwater from warm—perhaps poorly insulated—parts of sloped roofs may refreeze on colder areas or on the eaves and cause high ice loads and ice damming, water back-up under shingles, and icicles, which present a danger if they fall. These situations can be alleviated by taking steps to decrease heat loss from warm surfaces.

29. Since drainage under the snow cover on flat or nearly flat roofs is not generally as good as on those with slopes, meltwater, slush and ice may be retained longer. Also, snow accumulations near projections can melt as a result of heat loss through the roof, solar radiation or exhausted warm air. The resulting meltwater may migrate to the lower areas of the roof causing heavy loads. The centres of bays are particularly vulnerable if the drains are located at points of minimum deflection. This redistribution of load may cause further deflection and lead to an instability similar to that produced by rain ponding (see Commentary H).

Detailed Explanations of Accumulation Factors

30. **Basic roof shapes (NBC Articles 4.1.6.9. and 4.1.6.10.).** NBC Articles 4.1.6.9. and 4.1.6.10. apply to the basic roof shapes: simple flat and shed roofs, simple gable roofs, and simple arch and curved roofs and domes. More complex shapes can often be considered as combinations of these. Where the roofs depicted in these Articles are adjacent to higher roofs, have projections, or are combined to form valleys, NBC Articles 4.1.6.5. to 4.1.6.8., 4.1.6.11. and 4.1.6.12. should also be consulted.

31. **Gable, flat and shed roofs (NBC Articles 4.1.6.3. and 4.1.6.9.).** On gable roofs, both uniformly distributed and unbalanced loads should be considered for all slopes less than 70° (or 60° for unobstructed slippery roofs), as described in NBC Articles 4.1.6.3. and 4.1.6.9. For gable roofs with slopes equal to or less than 15°, the load distribution is determined by Case I of NBC Article 4.1.6.9., but is also subject to the general requirements of NBC Article 4.1.6.3. for full and partial loading, which now apply to the Case I loading only. For gable roofs with slopes greater than 15°, Case II of NBC Article 4.1.6.9., which accounts for unbalanced loading, and Case I both apply. Case II loading is intended to account for the blowing of snow from the windward over to the leeward side as well as the removal of snow due to sliding from one side, for example. Flat and shed (single-sloped) roofs are subject to Case I and full and partial loading only.

32. Arch roofs, curved roofs and domes (NBC Article 4.1.6.10.). Uniform and unbalanced load distributions are particularly important to consider in the design of arch and curved roofs and domes.[18][21][22] In addition, the requirements for full and partial loading apply.

33. Unbalanced load distributions caused by drifting snow and/or snow sliding off the surface on either side of an arch or domed roof occur regularly and should not be overlooked.[21]

34. Valleys in curved or sloped roofs (NBC Article 4.1.6.12.). In the design of roofs with valleys, uniform loads and loads accounting for drifting, sliding or creep, and the movement of meltwater are important to consider. A reduction factor due to slope is allowed for Case I loading because, as the snow creeps down the slope and wrinkles and layers at the bottom of the valley, the loads on the upper slopes are reduced. Since Cases II and III describe the worst loads due to drifting and slope effects, the C_s factor is taken as equal to 1.0.

35. Multiple-gable roofs with non-parallel roof lines. Section 1641.3 of the 1998 California Building Code[24] contains information regarding the potential buildup of snow at sloping valley lines created by non-parallel gable roof sections.

36. Multi-level roofs, obstructions and parapets (NBC Article 4.1.6.5.). Multi-level roofs, obstructions and parapets are all "bluff objects" creating turbulent wakes downwind where snow accumulates in drifts. Such objects can be considered as geometrical variations of a rectangular object situated on or adjacent to a lower flat roof. If the object is narrow and lower than the design depth of uniformly distributed snow on the roof, it is a "non-obstructing" object; if higher, it is considered as an obstruction; if higher than a non-obstructing bluff object and wide enough to accumulate a significant amount of snow on its upper surface, it is considered as an "upper level" roof.

37. The load of a snow drift on a roof that is adjacent to a higher one is taken to have a trapezoidal shape as illustrated in NBC Figure 4.1.6.5.-A. Thus, C_a varies with distance x from the step in roof elevation, starting at C_{a0} at $x = 0$ and decreasing linearly to a value $C_a(x_d)$ at the tail of the drift defined by $x = x_d$. The magnitude of the drift on the lower roof depends on the amount of snow that can drift from the upper roof or across the lower roof and be trapped in the step: accordingly, the larger the upper or lower roof, the greater the loading in the step.[6] For roofs with relatively low steps and, in particular, where the accumulation is primarily due to snow drifting from a large upper level roof (see Case I in NBC Article 4.1.6.5.), the snow accumulation can reach the top of the step. In this Case, C_{a0} is taken as follows:

$$C_{a0} = \beta \frac{\gamma h}{C_b S_s} \tag{2}$$

where
- h = difference in elevation between the lower roof surface and the top of the parapet on the upper roof,
- γ = specific weight of snow, and
- β = factor set to 1.0.

In the region of the drift, C_w is 1.0, since the lower roof is sheltered by the roof step. When snow drifts across the lower roof and into the step (see Cases II and III in NBC Article 4.1.6.5.), the step tends not to get filled right to the top. Therefore, $\beta = 0.67$ for these Cases.

However, where there is not sufficient snow to fill in the step, an upper limit on C_{a0}, calculated as follows, may be used:

$$C_{a0} = \frac{F}{C_b} \tag{3}$$

where
- F = factor specified in NBC Article 4.1.6.5. as a function of β, specific weight of snow, γ, characteristic length of the source area for snow, l_{cs}, parapet height, and ground snow load, S_s, and
- C_b = basic snow load factor for the lower roof.

The expression for l_{cs} in NBC Article 4.1.6.5. accounts for the roof-length-to-roof-breadth ratio. Where one dimension is much larger than the other, computer simulations[6] have shown that, due to the variability in wind direction, snow is unable to drift along the full length of the longer dimension into the step at the end, without much of it going off the sides. This limits the effective source area, and the effect is incorporated in NBC Article 4.1.6.5. through the use of l_{cs}.

Background information on the derivation of the method presented in NBC Article 4.1.6.5. is provided in Reference [6]. For buildings in sheltered locations (i.e., Exposure B, as defined in Commentary I), an upper limit of 5.0 is placed on the value of F since less drifting occurs in such locations. The horizontal length of the drift, x_d, extending out from the step is based on the top surface of the drift having a 1:5 slope.

Drifts deposited as a result of a change in elevation occur not only when the upper roof is part of the same building, but also when it is part of an adjacent building not more than 5 m away, as described in NBC Article 4.1.6.5. Where the upper roof is very large, the limiting gap, a, of 5 m should be confirmed by model tests. Where the drift obtained from NBC Article 4.1.6.5. is longer than the lower roof, the drift should be truncated at the edge of the lower roof.

Drifts can accumulate at the corners of multi-level roofs as a result of drifting from several directions. The drift shapes shown in NBC Article 4.1.6.8. are largely based on engineering judgement and serve to provide a common basis for design.

38. **Sample Calculation 1: Snow Drift Load on a Lower Roof.** A 3.2 m high penthouse is erected on a flat roof on a building in Ottawa; the roof has a 0.5 m high parapet along its perimeter. The calculation seeks to determine the snow drift load in Area A of the lower roof (see Figures G-1 to G-3).

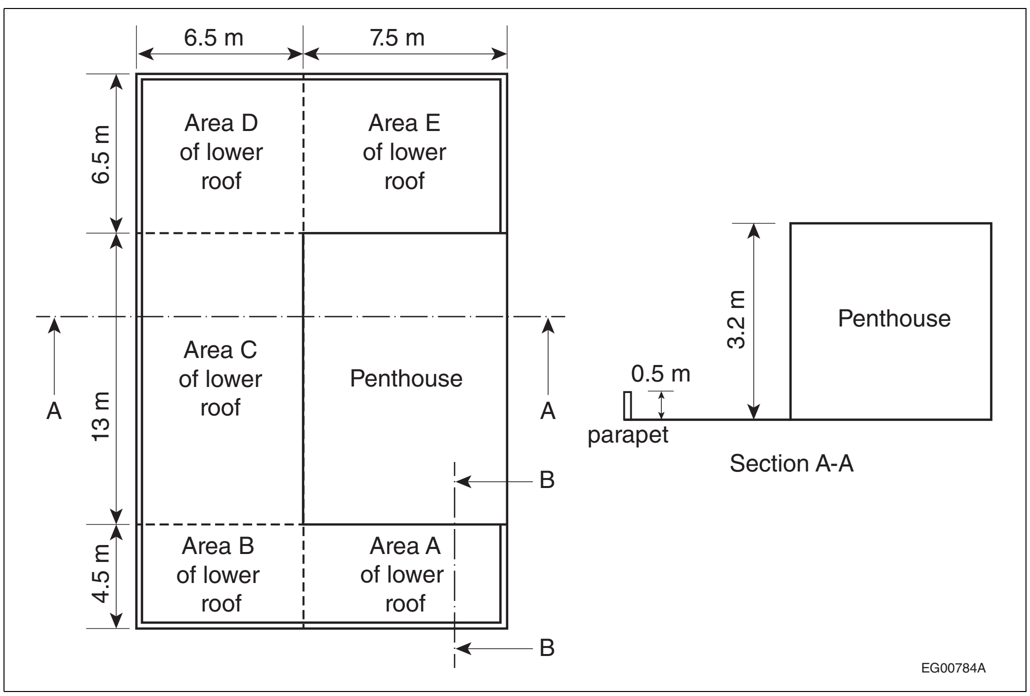

Figure G-1
Dimensions of roof areas for Sample Calculation 1

The following parameters are used to estimate the drift load on the lower roof:

C_w = 1.0,
C_s = 1.0 (flat roof),
C_b = 0.8,
I_s = 1 (Normal Importance Category building),
S_s = 2.4 kPa (NBC Table C-2 in Appendix C), and
S_r = 0.4 kPa (NBC Table C-2 in Appendix C).

Step 1 (NBC Sentence 4.1.6.13.(1)): Calculate γ using min ($0.43S_s + 2.2\ kN/m^3$, $4\ kN/m^3$)
$\gamma = 3.23\ kN/m^3$

Step 2 (NBC Sentence 4.1.6.5.(3)): Calculate l_{cs} using $2w_s - \frac{w_s^2}{l_s}$

Case I	Case II	Case III
l_s = 13 m w_s = 7.5 m l_{cs} = 10.67 m	l_s = 14 m w_s = 4.5 m l_{cs} = 7.55 m	l_s = 13 m w_s = 6.5 m l_{cs} = 9.75 m

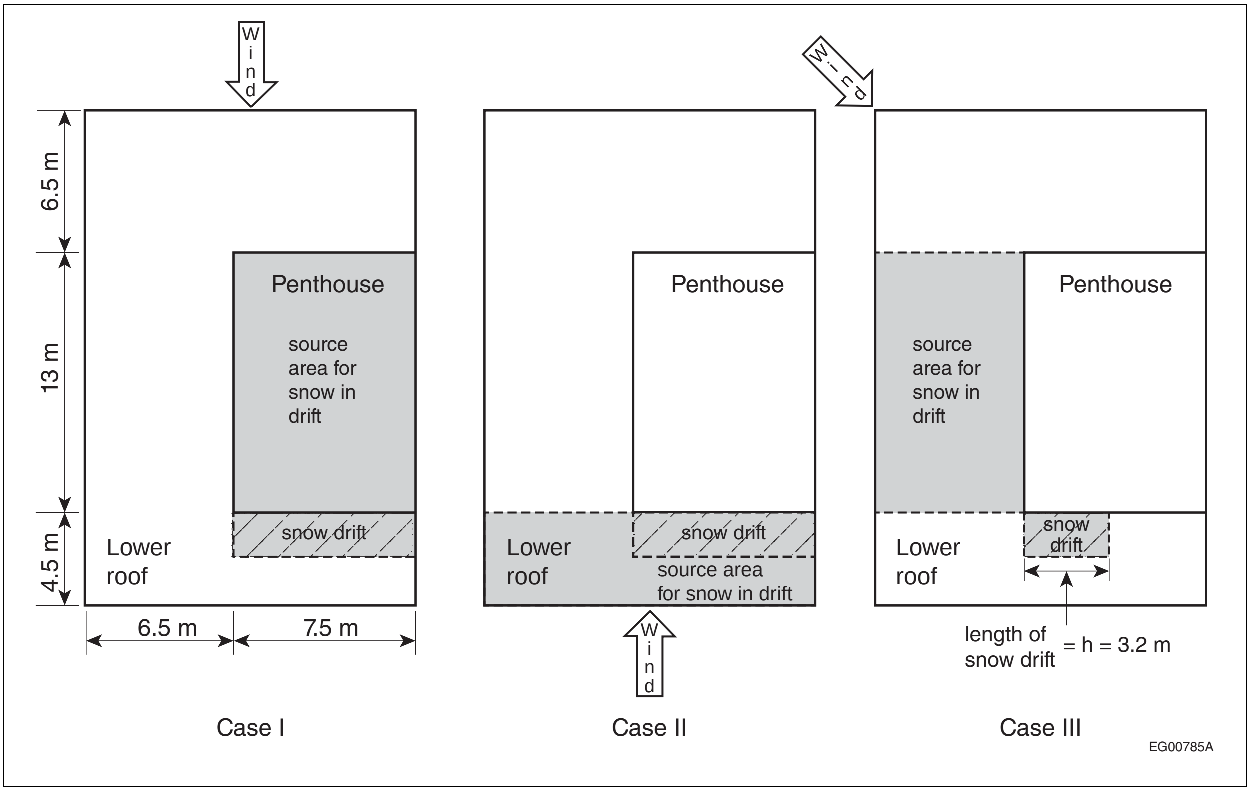

Figure G-2
Snow load cases for lower roof (Sample Calculation 1)

Step 3 (NBC Sentence 4.1.6.5.(3)): Calculate h'_p using $h_p - \frac{0.8S_s}{\gamma}$ but $0 \le h'_p \le \frac{l_{cs}}{5}$

Case I	Case II	Case III
$h_p = 0$ $h'_p = 0 - \dfrac{0.8 \times 2.4}{3.23} = -0.58 < 0$ $h'_p = 0$	$h_p = 0.5\,m$ $h'_p = 0.5 - 0.58 = -0.08 < 0$ $h'_p = 0$	$h_p = 0.5\,m$ $h'_p = 0.5 - 0.58 = -0.08 < 0$ $h'_p = 0$

Step 4 (NBC Sentence 4.1.6.5.(3)): Calculate F using $0.35\beta\sqrt{\frac{\gamma(l_{cs} - 5h'_p)}{S_s}} + C_b$

Case I	Case II	Case III
$F = 0.35 \times 1\sqrt{\frac{3.23\times10.67}{2.4}} + 0.8$ $F = 2.13$	$F = 0.35 \times 0.67\sqrt{\frac{3.23\times7.55}{2.4}} + 0.8$ $F = 1.55$	$F = 0.35 \times 0.67\sqrt{\frac{3.23\times9.75}{2.4}} + 0.8$ $F = 1.65$

Step 5 (NBC Sentence 4.1.6.5.(3)): Calculate C_{a0} using $\min\left(\beta\frac{\gamma h}{C_b S_s}, \frac{F}{C_b}\right)$

Case I	Case II	Case III
$C_{a0} = \min\left(\frac{3.23\times3.2}{0.8\times2.4}, \frac{2.13}{0.8}\right)$ $C_{a0} = \min(5.38, 2.66) = 2.66$	$C_{a0} = \min\left(0.67\times\frac{3.23\times3.2}{0.8\times2.4}, \frac{1.55}{0.8}\right)$ $C_{a0} = \min(3.6, 1.94) = 1.94$	$C_{a0} = \min\left(0.67\times\frac{3.23\times3.2}{0.8\times2.4}, \frac{1.65}{0.8}\right)$ $C_{a0} = \min(3.6, 2.06) = 2.06$
C_{a0} = max(2.66,1.94,2.06) = 2.66		

Step 6 (NBC Sentence 4.1.6.5.(2)): Calculate x_d using $5\frac{C_b S_s}{\gamma}(C_{a0} - 1)$

$$x_d = 5 \times \frac{0.8 \times 2.4}{3.23}(2.66 - 1) = 4.93\,\text{m}$$

Step 7 (NBC Figure 4.1.6.5.-A): Calculate h' using $h - \frac{C_b C_w S_s}{\gamma}$

$$h' = 3.2 - \frac{0.8 \times 1 \times 2.4}{3.23} = 2.6\,\text{m}$$

Step 8 (NBC Figure 4.1.6.5.-A): Calculate x using $10h'$

$$x = 10 \times 2.6 = 26\,\text{m}$$

Step 9 (NBC Sentence 4.1.6.5.(1)): Calculate C_a using $C_{a0} - (C_{a0} - 1)\frac{x}{x_d}$ for $0 \le x \le x_d$ and $C_a = 1$ for $x > x_d$

$$C_a = 2.66 - (2.66 - 1)\frac{x}{4.93} \text{ for } 0\,\text{m} \le x \le 4.93\,\text{m}$$
$$C_a = 1 \text{ for } x > 4.93\,\text{m}$$

Step 10 (NBC Clause 4.1.6.7.(1)(a)): Calculate the parapet's effect on the lower roof using $C_{a0} = \min\left(\frac{0.67\gamma h}{C_b S_s}, 1 + \frac{\gamma l_0}{7.5 C_b S_s}\right)$

$$C_{a0} = \min\left(\frac{0.67 \times 3.23 \times 0.5}{0.8 \times 2.4}, 1 + \frac{3.23 \times 14}{7.5 \times 0.8 \times 2.4}\right)$$
$$C_{a0} = \min(0.56, 4.14) = 0.56 < 1$$

The parapet's effect is not significant and should therefore be ignored.

Step 11 (NBC Sentence 4.1.6.2.(1)): Calculate distribution of snow drift loads, S, in Area A (Section B-B of Figure G-1) using $I_s[S_s(C_bC_wC_sC_a) + S_r]$. Based on the results of Step 5, the drift area corresponds to Case I. Therefore:

$$S = 1 \times [2.4\,(0.8 \times 1 \times 1 \times 2.66) + 0.4] = 5.5\,\text{kPa next to the penthouse}$$
$$S = 1 \times [2.4\,(0.8 \times 1 \times 1 \times 1.15) + 0.4] = 2.6\,\text{kPa at 4.5 m away from the penthouse}$$

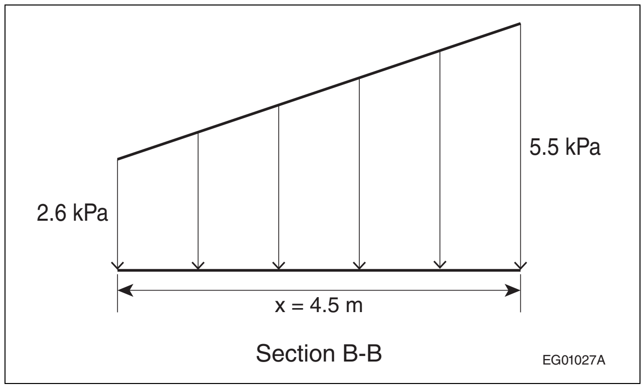

Figure G-3
Snow drift loads in Area A of lower roof (Section B-B of Figure G-1)

The same calculations are carried out for snow drifts in Areas C and E. The drifts at corners in Areas B and D are calculated in accordance with NBC Sentence 4.1.6.8.(1).

39. **Sample Calculation 2: Gap Effect on Snow Drift Load.** Two administrative buildings located in Dorval, Quebec are 2 m apart and have flat roofs with an elevation difference of 3.2 m. The lower roof is 24 m long and 14 m wide, and the higher roof is 16 m long and 14 m wide. The calculation seeks to determine the snow drift load in Area A of the lower roof (see Figures G-4 to G-6).

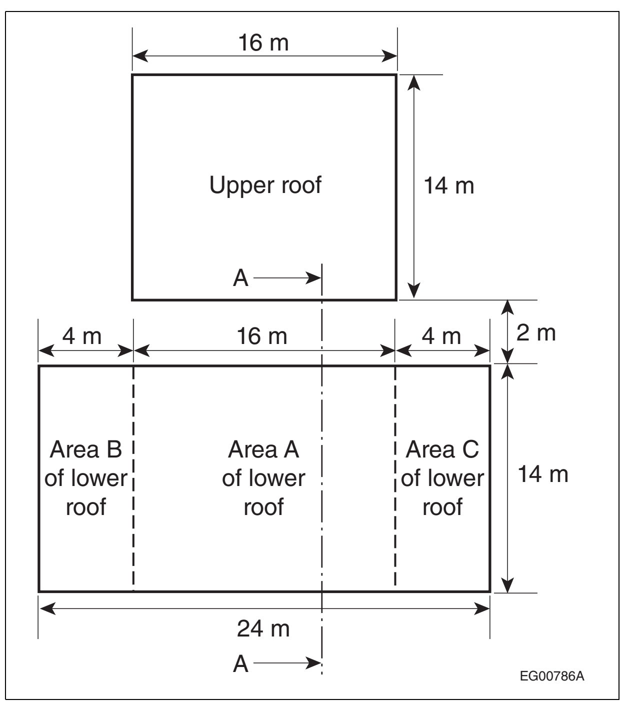

Figure G-4
Dimensions of roof areas for Sample Calculation 2

The following parameters are used to estimate the gap effect on the snow drift load on Area A of the lower roof:

C_w = 1.0,
C_s = 1.0 (flat roof),
C_b = 0.8,
I_s = 1 (Normal Importance Category building),
S_s = 2.4 kPa (NBC Table C-2 in Appendix C), and
S_r = 0.4 kPa (NBC Table C-2 in Appendix C).

Step 1 (NBC Sentence 4.1.6.13.(1)): Calculate γ using min ($0.43S_s + 2.2$ kN/m³, 4 kN/m³)
γ = 3.23 kN/m³

Step 2 (NBC Sentence 4.1.6.5.(3)): Calculate l_{cs} using $2w_s - \frac{w_s^2}{l_s}$

Case I	Case II	Case III
l_s = 16 m w_s = 14 m l_{cs} = 15.75 m	l_s = 24 m w_s = 14 m l_{cs} = 19.83 m	n/a

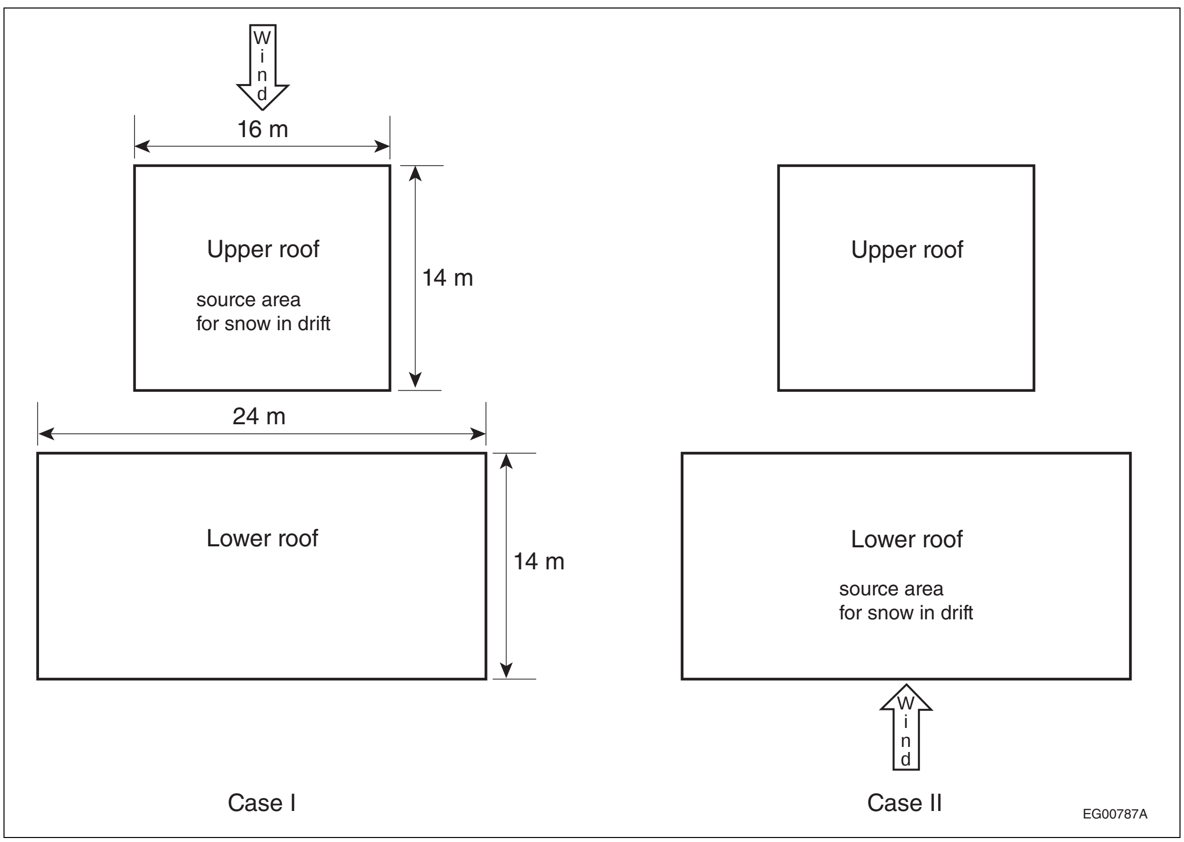

Figure G-5
Snow load cases for lower roof (Sample Calculation 2)

Step 3 (NBC Sentence 4.1.6.5.(3)): Calculate h_p' using $h_p - \frac{0.8S_s}{\gamma}$ but $0 \le h_p' \le \frac{l_{cs}}{5}$

Case I	Case II
$h_p = 0$ $h'_p = 0 - \dfrac{0.8 \times 2.4}{3.23} = -0.58 < 0$ $h'_p = 0$	$h_p = 0$ $h'_p = 0$

Step 4 (NBC Sentence 4.1.6.5.(3)): Calculate F using $0.35\beta\sqrt{\frac{\gamma(l_{cs}-5h'_p)}{S_s}} + C_b$

Case I	Case II
$F = 0.35 \times 1\sqrt{\dfrac{3.23 \times 15.75}{2.4}} + 0.8 = 1.61$	$F = 0.35 \times 0.67\sqrt{\dfrac{3.23 \times 19.83}{2.4}} + 0.8 = 2.01$

Step 5 (NBC Sentence 4.1.6.5.(3)): Calculate C_{a0} using $\min\left(\beta\frac{\gamma h}{C_b S_s}, \frac{F}{C_b}\right)$

Case I	Case II
$C_{a0} = \min\left(\dfrac{3.23 \times 3.2}{0.8 \times 2.4}, \dfrac{1.61}{0.8}\right)$ $C_{a0} = \min(5.38, 2.01) = 2.01$	$C_{a0} = \min\left(0.67 \times \dfrac{3.23 \times 3.2}{0.8 \times 2.4}, \dfrac{2.01}{0.8}\right)$ $C_{a0} = \min(3.6, 2.51) = 2.51$
C_{a0} = max (2.01, 2.51) = 2.51	

Step 6 (NBC Sentence 4.1.6.5.(2)): Calculate x_d using $5\frac{C_b S_s}{\gamma}(C_{a0} - 1)$

$$x_d = 5 \times \frac{0.8 \times 2.4}{3.23}(2.51 - 1) = 4.49\,\text{m}$$

Step 7 (NBC Figure 4.1.6.5.-A): Calculate h′ using $h - \frac{C_b C_w S_s}{\gamma}$

$$h' = 3.2 - \frac{0.8 \times 1 \times 2.4}{3.23} = 2.6\,\text{m}$$

Step 8 (NBC Figure 4.1.6.5.-A): Calculate x using 10h′

$$x = 10 \times 2.6 = 26\,\text{m}$$

Step 9 (NBC Sentence 4.1.6.5.(1)): Calculate C_a using $C_{a0} - (C_{a0} - 1)\frac{x}{x_d}$ for $0 \le x \le x_d$ and $C_a = 1.0$ for $x > x_d$

$$\begin{aligned} C_a &= 2.51 - (2.51 - 1)\frac{x}{4.49} \text{ for } 0\,\text{m} \le x \le 4.49\,\text{m} \\ \therefore C_a &= 2.51 \text{ for } x = 0\,\text{m and } 1.84 \text{ for } x = 2\,\text{m} \\ C_a &= 1 \text{ for } x > 4.49\,\text{m} \end{aligned}$$

Step 10 (NBC Sentence 4.1.6.2.(1)): Calculate distribution of snow drift loads, S, in Area A (Section A-A of Figure G-4) using $I_s[S_s(C_bC_wC_sC_a) + S_r]$

$$\text{Where } x = 0\,\text{m}, S = 1 \times [2.4\,(0.8 \times 1 \times 1 \times 2.51) + 0.4] = 5.22\,\text{kPa}$$
$$\text{Where } x = 2\,\text{m}, S = 1 \times [2.4\,(0.8 \times 1 \times 1 \times 1.84) + 0.4] = 3.93\,\text{kPa}$$
$$\text{Where } x > 4.49\,\text{m}, S = 1 \times [2.4\,(0.8 \times 1 \times 1 \times 1) + 0.4] = 2.32\,\text{kPa}$$

The drifts at corners in Areas B and C are calculated in accordance with NBC Article 4.1.6.8.

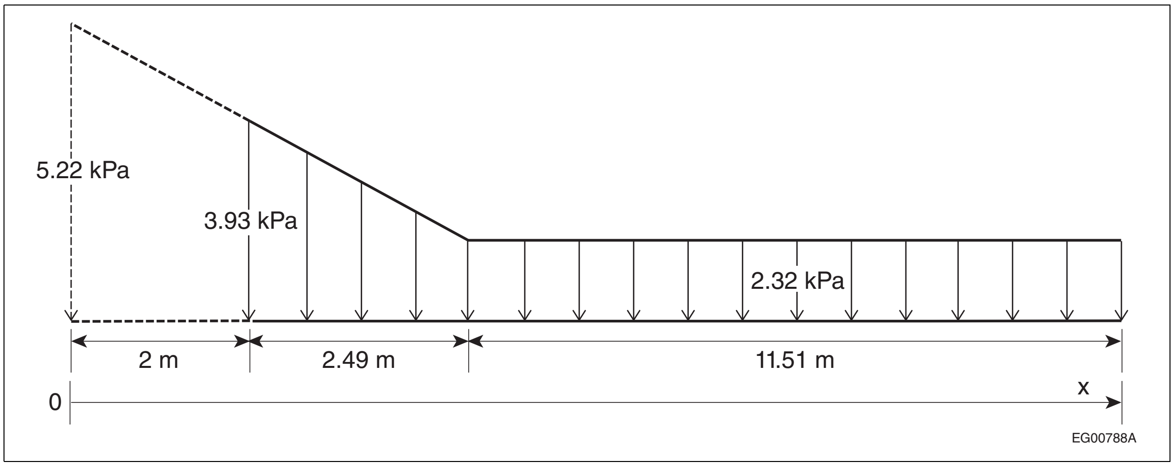

Figure G-6
Snow drift loads in Area A of lower roof (Section A-A of Figure G-4)

40. **Canopies or small roofs adjacent to tall buildings.** Where the lower roof is small relative to the difference in elevation between two adjacent roofs—such as in the case of an entrance canopy at the base of a high-rise building—the loads will be less than those described in NBC Article 4.1.6.5. because the snow from the upper roof is dispersed over a wide area and drifting over the small lower roof is insufficient to build up a significant accumulation. While insufficient research has been carried out to fully evaluate the reduced loadings, the following approach is suggested. For small area lower roofs with a plan area less than 25 m² that are situated more than 20 m below the upper level roof, C_a may be taken as 1.0. Where the height difference, h, is less than 10 m, C_a is determined in accordance with NBC Article 4.1.6.5. For cases between h = 10 m and h = 20 m, the form of drift described in NBC Article 4.1.6.5. is used but with C_{a0} reduced in linear fashion as h varies from 10 to 20 m as follows:

$$C_{a0} = 1.0 + \left[\left(\frac{(20 - h)}{10}\right)\left(\frac{F}{C_b} - 1.0\right)\right] \tag{4}$$

where C_b is the basic roof snow load factor applicable to the lower roof.

41. For buildings on sheltered sites, the wind exposure factor, C_w, is 1.0 for all areas of the lower roof. For Low and Normal Importance Category buildings on exposed sites, C_w on the lower roof should still be taken as 1.0 within a region sheltered by the step extending outwards from the step a distance equal to 10 h′, where h′ is the difference in elevation between the top of the upper roof's parapet and the snow surface on the exposed portion of the lower roof. This definition implies that

$$h' = h - \frac{C_bC_wS_s}{\gamma} \tag{5}$$

where the applicable value of C_w is either 0.75 or 0.5, as provided for in NBC Sentence 4.1.6.2.(4), and C_b is the appropriate value for the exposed portion of the roof, i.e., normally 0.8 but may be higher for large roofs, as specified in NBC Clauses 4.1.6.2.(2)(a) and (b).

42. **Multi-level roofs with a sloped upper roof (NBC Figure 4.1.6.11.).** A lower roof is designed for the loads addressed in NBC Article 4.1.6.5. plus the potential additional load resulting from snow sliding from the upper roof, as described in NBC Article 4.1.6.11. Because of the low probability that both the upper and lower roofs will be supporting the design snow load over their entire areas simultaneously when sliding occurs, the lower roof is assumed to carry its full load, determined in accordance with NBC Article 4.1.6.5., plus 50% of the total weight of the Case I snow load presented in NBC Article 4.1.6.9. from the portion of the upper roof that slopes toward the lower roof. The load is distributed based on the relative sizes, slopes and positions of the two roofs. If all the sliding snow cannot be retained on the lower roof because it is too small, appropriate reductions in snow load may be made. A profile of the snow depth on the lower roof should be drawn to confirm that the loading is reasonable. The amount of snow that can be expected to slide from the higher roof to the lower one will depend on the friction coefficient of the roofing membrane on the upper roof. Snow can slide even from a roof with a very low slope if the surface is slippery; therefore NBC Sentence 4.1.6.11.(1) places no lower limit on roof slope.

    NBC Sentence 4.1.6.11.(2) states that, where a parapet or other effective means are incorporated into the design of the upper roof to retain the snow and not allow it to slide to the lower roof, the additional sliding load on the lower roof need not be considered. Retention effectiveness will depend on the selected design approach: e.g., snow guards, parapet walls, bar barriers, cable barriers, plates, general roughness. In selecting the appropriate approach, consideration will need to be given to roof slope and exposure to wind and sun in conjunction with snow avalanche, snow creep, impact loads due to snow and ice slides, and freeze/thaw cycling, which creates multiple slip layers. It is incumbent upon the designer or supplier of the retention product to provide sufficient evidence of its effectiveness at retaining design level snow accumulations given the intended application. In some cases, in-situ evidence or efficacy determined from cold-room mock-up testing is useful. Note that, if the upper sloped roof incorporates retention measures, then snow load reductions cannot be applied to it.

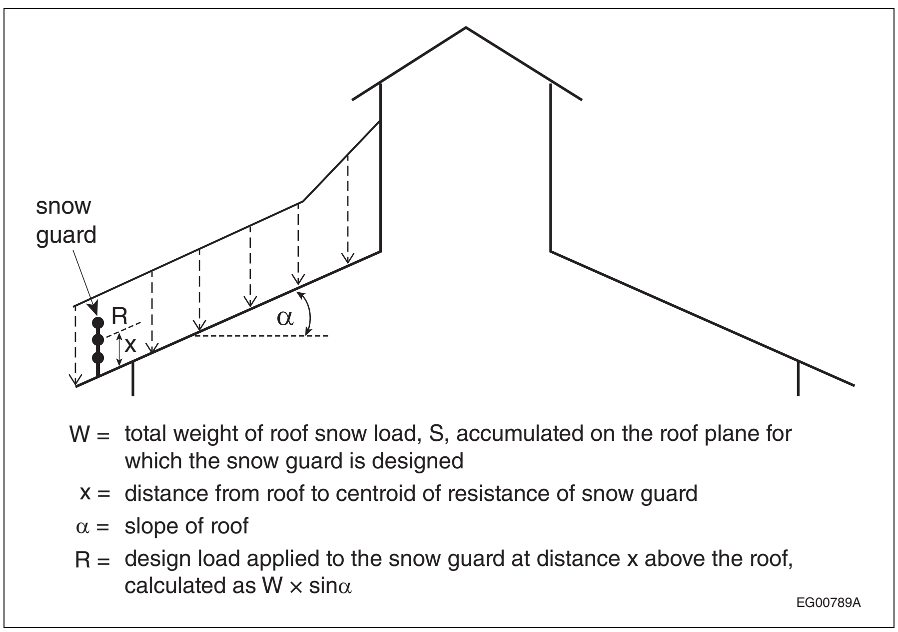

Figure G-7
Design of snow guard on sloped roof

Snow guards (also referred to as snow fences) should only be assumed to be able to restrain a maximum accumulation of snow equal to their height when considering the snow load on the lower roof. If the design snow accumulation is higher than the guard, the portion of snow accumulation that is unrestrained by the guard may slide off the upper roof and onto the lower roof. For example, a thin layer of ice can develop on the snow between snow falls, creating a slippery plane. Whether a snow guard reaches the design snow height or not, it—along with its connections—should be designed for the potential forces exerted by the full design snow load, assuming all snow is restrained. This also includes snow on both sides of the guard. Gutters and other obstructions installed to prevent snow from sliding should also be designed similarly. Figure G-7 illustrates how the design loads for roof guards are determined.

For a guard to be considered effective in retaining snow, it must be constructed of horizontal rails with a minimum diameter of 25 mm spaced no more than 100 mm apart.

43. **Areas adjacent to roof projections (NBC Article 4.1.6.7.).** NBC Article 4.1.6.7. addresses trapezoidal snow drift loads abutting significant vertical roof projections, such as elevator, air-conditioning and fan housings, small penthouses and wide chimneys, which are caused by snow drifting across the roof. In previous editions of this Commentary, such drifts were assumed to reach a height equal to 0.67h, h being the height of the projection, and to extend a distance of 2h out from the projection. In the 2015 edition of the NBC, the slope of the drift's surface is taken to be the same as for roof steps, i.e., 1 in 5, and in addition to the 0.67h limit on drift height, the drift height is also limited to be less than $\left(\frac{C_b S_s}{\gamma} + \frac{1}{7.5} l_0\right)$, where l_0 = the longest horizontal dimension of the projection. This latter limit reflects the difference between roof projections and roof steps. See Figure G-8.

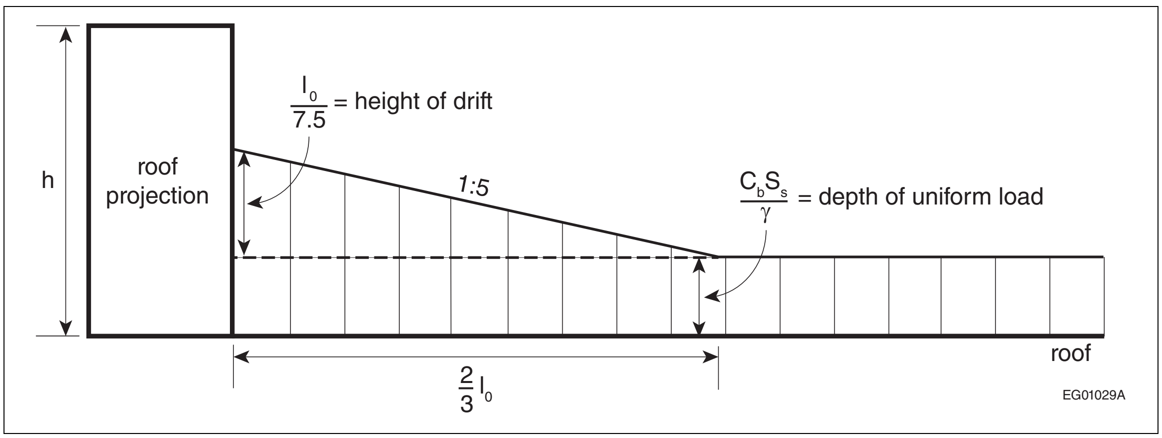

Figure G-8
Depth of snow drift adjacent to a roof projection

Roof projections are more affected by changes in wind direction since areas of aerodynamic shade easily become areas of scour with a change in direction. This tends to limit the length of the drifts. It is assumed that drifts can be no longer than $\frac{2}{3}l_0$ and this, combined with the 1 in 5 slope assumption, results in the second limit on drift height, $\left(\frac{C_b S_s}{\gamma} + \frac{1}{7.5} l_0\right)$. NBC Article 4.1.6.7. allows the lower of this and the 0.67h limit to be used and provides a simplified approach to calculating drift loads around projections. However, with higher values of l_o, this simplified approach may result in higher loads than if the area around the projection were treated as a lower level roof. As such, NBC Sentence 4.1.6.7.(2) allows drifts at the base of large projections to be calculated the same way as those for a roof step, i.e., in accordance with NBC Article 4.1.6.5.

Snow drifts around projections that are less than 3 m long are permitted to be ignored as they will not have a significant impact on the structure.

44. **Effect of solar panels on snow loads.** Solar collectors can affect the distribution of snow at changes in roof elevation. Of particular concern are larger arrays wherefrom sliding and melting snow can result in snow and ice buildup at their base, causing local overloading of the structure in the area of the array's support. These effects and the configuration of the solar collectors should be taken into consideration when developing design loads for new structures or assessing the feasibility of a retrofit installation.

Unusual Roofs

45. Snow loads are difficult to predict in some cases, particularly for unusually shaped roofs, exceptionally large roofs, and roofs over which the airflow is significantly affected by other buildings or topographic features. In such cases, the designer should calculate and plot the snow depths to scale applying the appropriate specific weight of snow to judge whether the distributions look

reasonable. In some circumstances, wind tunnel or water flume tests will be beneficial to assist in the evaluation. Methods for these types of testing for snow drift loads and the associated analysis are described in ASCE/SEI 49.

Parking Decks

46. Roofs used as parking decks should be designed for either the uniform live loads specified in NBC Table 4.1.5.3., the concentrated loads specified in NBC Table 4.1.5.9., or the roof snow and rain loads prescribed in NBC Subsection 4.1.6., whichever produces the most critical effect in the members concerned. Where snow removal occurs from time to time, consideration should be given to the loads due to snow removal equipment and to the weight of piled snow.

Roofs with an Occupancy with Snow Removal

47. Roofs with an occupancy, other than parking decks, from which snow is to be removed from time to time, should be designed for the live loads prescribed for the intended use in accordance with NBC Subsection 4.1.5. or for the roof snow and rain loads prescribed in NBC Subsection 4.1.6., whichever produces the most critical effect in the members concerned.

Roofs with an Occupancy without Snow Removal

48. Roofs with an occupancy whose load is expected to be maintained throughout the winter and wherefrom snow will not be removed should be designed for the effects of snow load and live load acting simultaneously by applying the load combination factors of NBC Table 4.1.3.2.-A. An automobile dealership with cars stored on the roof is an example of a situation where both loads may act simultaneously.

Roofs without an Occupancy

49. Roofs without an occupancy should be designed for either the snow and rain loads prescribed in NBC Subsection 4.1.6., the minimum live load of 1.0 kPa prescribed in NBC Table 4.1.5.3., or the minimum concentrated load of 1.3 kN prescribed in NBC Table 4.1.5.9., whichever produces the most critical effect in the members concerned.

Sunshades

50. Sunshades consisting of a grillage of metal slats are becoming more common on buildings. They should be designed for snow and ice loads. Where the horizontal gaps between the slats are 100 mm or less, the snow can bridge the gaps and the sunshade should be considered as solid from the standpoint of snow loading. Where snow and ice can slide from a sloped roof above the sunshade, even gaps that are larger than 100 mm may make the sunshade behave as a solid surface.

Full and Partial Loading

51. All roof areas, including those to be designed for increased or decreased loads according to NBC Articles 4.1.6.5. to 4.1.6.12., must be designed for the full specified load given in NBC Article 4.1.6.2. applied over the entire area. However, only flat and shed roofs, low-sloped gable roofs addressed in NBC Article 4.1.6.9., and arch roofs, curved roofs and domes addressed in NBC Article 4.1.6.10. need to be designed for Case I loading distributed on one portion of the area and half of this on the remainder of the area, the location and size of such partial areas being chosen to give the most critical effects in the members and joints concerned. These requirements do not imply checkerboard loading because the probability that checkerboard loading will occur to a degree that will cause the worst conditions for supporting members is generally too remote to be considered in design.[25] On many types of roofs like the ones considered in NBC Articles 4.1.6.9. and 4.1.6.10., a number of separate cases of full and partial loading will be required to ensure the proper design of all elements.

52. The reason for these requirements is that snow seldom accumulates according to the simple configurations in NBC Articles 4.1.6.9. and 4.1.6.10. Consequently, full and partial loading must be considered for the design of structural members that are sensitive to changes in load distribution (e.g., truss diagonals and cantilevers) and that would not otherwise be designed for unbalanced loads.

Snow Removal

53. Although it is fairly common practice in some areas of Canada to remove snow from roofs after heavy snowfalls, NBC Article 4.1.6.14. does not allow a reduction of the design load to account for this because:
   (a) snow removal cannot be relied upon (experience in several countries has shown that during and after extreme snowstorms, traffic is immobilized and snow removal crews are either unavailable due to high demand or unable to access certain areas),
   (b) snow cannot be effectively removed from the centre of large roofs, and
   (c) adverse unbalanced loading can occur as a result of certain patterns of snow removal.

54. NBC Article 4.1.6.14. also does not allow design snow loads to be reduced based on the installation of melting systems, which periodically clear a roof of snow, because adequate energy for melting may not be available when required. Furthermore, as the years pass, the importance of keeping the system functioning (perhaps at great cost) may be forgotten.

Ice Loading on Structures

55. Loads due to ice accretion on the exposed surfaces of superstructure members, railings, lattice towers and signs are described in CSA S6, "Canadian Highway Bridge Design Code," and in CSA S37, "Antennas, Towers, and Antenna-Supporting Structures." Environment and Climate Change Canada has a model to compute ice loading on vertical and horizontal surfaces and cables, which is based on climate data at weather stations. For such structures, NBC Article 4.1.6.15. refers designers to CSA S37.

Minimum Roof Live Load

56. NBC Articles 4.1.5.3. and 4.1.5.9. provide for a minimum uniform roof live load of 1 kPa and a minimum concentrated live load of 1.3 kN. These live loads are "use and occupancy loads" intended to provide for maintenance loadings, workers and so forth. These live loads are not reduced as a function of area or as a function of the roof slope.

History of Snow Load Provisions in the NBC

57. In the 1953 edition of the NBC, design snow loads were equal to the ground snow load, with reductions allowed for sloped roofs only. The load values were very approximate and resulted in over-design for some roofs and under-design for others, particularly in areas subject to high drift loads. Information on which to base a more refined assessment of the loads was not available until a countrywide survey of snow loads on roofs was undertaken by the Institute for Research in Construction of the National Research Council of Canada (now called NRC Construction) with the help of many volunteer observers. This survey provided evidence on the relationship between ground and roof loads and enabled the committees responsible for the 1960 edition of the NBC to adjust the Code requirements. The roof load was set at 80% of the ground load, which was based on a return period of 30 years and adjusted to allow for the increase in the load caused by rainwater absorbed by the snow.

58. With the introduction of the 1965 Code and the Commentary on NBC Part 4, further changes made by the Revision Committee on Structural Loads and Procedures led to a more rational approach to design loads. The Committee concluded that all roof loads were directly related to the snow load on the ground; consequently, the roof snow loads were removed from the table of Design Data for Selected Locations in Canada. The basic design load remained at 80% of the ground load, except that a snow load of 60% of the ground load was allowed for roofs exposed to the wind. This reduction was made because, at the same time, allowance was made for a variety of influences causing the accumulation of snow on roofs. This was done by means of snow load coefficients or accumulation factors, which were shown in the form of simple formulas and diagrams similar to those in the 2015 NBC Articles 4.1.6.5. to 4.1.6.12. In addition, the slope reduction formula was changed from the step function used in 1960 to a linear function.

59. In the 1970 Code and Commentary, minor changes were made to the provisions for gable and arch roofs and more severe "full and partial loading provisions"—"full and zero loading" rather than "full and half"—were introduced.

60. In the 1975 Code and Commentary, few changes were made, except that the requirement for full and partial loading was considered too severe at “full and zero” and was changed back to “full and half” loading.

61. In the 1977 and 1980 Commentaries, the provisions for loads on arch roofs were changed and a number of rationalizations were made to help Code users better understand snow loads on roofs.

62. The 1985 Code and Commentary provisions were rewritten to simplify the presentation and to clarify the intent of the minimum roof loading of 1.0 kPa. Furthermore, the minimum roof loading was made independent of slope, the specific weight of snow on roofs was increased by 1.9% to give γ = 2.4 kN/m^3, full and partial loading was restricted to Case I loadings on buildings like the ones shown in the 2015 NBC Articles 4.1.6.9. and 4.1.6.10., and the unbalanced loading on arches was simplified.

63. In the 1990 Code and Commentary, a new slope reduction formula was given for unobstructed slippery sloped roofs, the specific weight of snow on roofs was increased to γ = 3.0 kN/m^3, the need for unbalanced snow loads on domes was emphasized, the minimum C_w was reduced to 0.5—rather than 0.75—for exposed roofs north of the treeline, and design roof snow loads were separated into snow and rain components consistent with the ground snow loads given in Chapter 1 of the 1990 Supplement.

64. In the 1995 Code and Commentary, new formulae were given for the accumulation factor, C_a, for the calculation of uniformly distributed snow loads on large flat upper or lower roofs. Additional information was also provided for snow loads on lower roofs and elongated spike drifts on high-low roof configurations.

65. In 2005, the return period for snow loads was increased from 30 years to 50 years and an importance factor for snow loads was introduced. The increase in loading on large-area roofs, which was previously captured in the accumulation factor, C_a, was taken out of C_a and incorporated into the basic snow load factor, C_b. The provisions for unbalanced loads on arch roofs were modified to apply to arches with a height-to-width ratio, h/b, as low as 0.05, compared with the previous limit of 0.10.

66. For the 2010 edition, only minor changes were introduced, one being the clarification of the drift loads around corners of upper level roofs and roof projections.

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Commentary H

Rain Loads

Commentary H

Rain Loads

1. In accordance with NBC Sentence 4.1.6.4.(1), any roof that can accumulate water must be designed for the load that results from a one-day rainfall on the horizontal projected area of the roof. This requirement applies whether or not the surface is provided with drainage, such as rainwater leaders. The distribution of rain load should be determined by the designer, who should take into account the shape of the roof, including camber, with or without creep deflection due to dead load, and also deflection due to rain.

2. Notwithstanding the above requirement, it is considered good practice when locating roof drains to take into account not only the roof slope but also deflection of the roof due to creep, snow and rain. Drains should be provided with suitable devices to prevent clogging by leaves or, where appropriate, suitable overflows should be provided through parapet walls.

3. In some areas of Canada, there is potential for the primary drainage system for a roof to become blocked due to freeze-thaw conditions. Roofs in these areas should be designed accordingly.

Ponding Instability

4. If a flat roof is too flexible, rainwater will not accumulate evenly over the roof but will flow to form ponds in a few local areas. This may lead to an instability similar to buckling, which can result in failure of the roof due to local overloading. In the case of one-way roof beams or decking simply supported on rigid supports, ponding instability will occur when the beam or decking stiffness is less than EI_{crit} given by

$$EI_{crit} = \rho g S \left(\frac{L}{\pi}\right)^4 \quad (1)$$

where
- E = modulus of elasticity,
- I = moment of inertia of the beam or decking,
- L = span,
- S = spacing of the beam or decking,
- ρ = mass density of water, kg/m³.

5. In the case of a two-way system of roof joists on girders, the critical stiffness can be approximated by

$$\frac{EI_{jcrit}}{EI_j} + \frac{EI_{gcrit}}{EI_g} = 1 \quad (2)$$

where EI_{jcrit} and EI_{gcrit} are given by Equation (1) for joists and girders, respectively.

6. Even if the roof system is stiffer than the critical values determined by Equations (1) and (2), calculated moments and deflections may be amplified due to ponding effects. A practical criterion is to require roof stiffness to be at least twice the critical stiffness. In the case of a one-way system on rigid supports, in terms of existing deflection requirements, this can be expressed as follows:

$$w > 15.4L\left(\frac{\Delta}{L}\right)_{allowable} \quad (3)$$

where w is the design load, in kPa, specified for deflection calculation, and $(\Delta/L)_{allowable}$ is the allowable deflection to span ratio (see Table D-1 of Commentary D). If, for a one-way system, the design load, w, is less than the critical value given in Table H-1, the effects of ponding should be considered. This applies particularly to large flat roofs in areas of heavy rainfall. Further information is given in References [1] to [7].

Table H-1
Critical Values of w for Ponding for a One-Way System (Equation (3))

Deflection/Span Ratio	w, kPa			
	L = 5 m	L = 10 m	L = 20 m	L = 30 m
1:180	0.43	0.86	1.71	2.57
1:240	0.32	0.64	1.28	1.93

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Commentary I

Wind Load and Effects

Commentary I

Wind Load and Effects

Notable Changes in the National Building Code of Canada (NBC) 2015

- Relocation of Commentary material to the NBC
- Clarification of the Static, Dynamic and Wind Tunnel Procedures for determining wind loads in NBC Article 4.1.7.1.
- Introduction of a topographic factor, C_t, which was previously considered as part of the exposure factor, C_e
- Elimination of wind direction as a factor in the calculation of wind loads on roof and wall claddings
- Introduction of provisions regarding exterior ornamentations, equipment and appendages
- Introduction of provisions on the Wind Tunnel Procedure

Notable Changes in this Commentary

Discussions on the following subjects have been added:

- solar arrays mounted on roofs
- limitations of computational fluid dynamics
- design basis for glass using ASTM E 1300 and CAN/CGSB-12.20-M
- wake buffeting and channeling effects
- comparison of earthquake and wind load risks and probability of failure

Wind Load Calculation Procedures

1. Three procedures for determining design wind load on buildings are indicated in NBC Subsection 4.1.7.: Static, Dynamic and Wind Tunnel.

2. The Static Procedure is appropriate for most wind load calculation cases, including for the design of the main structural system of most low- and mid-rise buildings as well as for the design of cladding on all buildings. The main structural system is an assemblage of structural elements that provides support and stability to the building as a whole. The system generally receives wind loading from more than one surface. The structure or element to be designed in these cases is relatively rigid. As such, detailed knowledge of the dynamic properties of these structures or elements is not required and dynamic actions of the wind can be represented by equivalent static loads.

3. The Dynamic Procedure is mainly intended for determining the overall effects of wind, including resonant response, on the main structural system of tall buildings and slender or long-span structures (with a frequency less than 1 Hz), but not on cladding and secondary structural members.[1] Its format is the same as that of the Static Procedure, except that the gust effect factor, C_g, and the exposure factor, C_e, are determined differently. C_g is derived from a series of calculations involving the following variables:
   (a) the intensity of wind turbulence for the site as a function of height and of the surface roughness of the surrounding terrain, and
   (b) the properties of the building, such as height, width, natural frequency of vibration, and damping.

   When multiplied by the reference velocity pressure, q, the importance factor, I_w, the exposure factor, C_e, and the pressure coefficient, C_p, the gust effect factor is expected to result in a static design pressure that represents the same peak load effect as the dynamic resonant response to the actual turbulent wind. In addition to the calculation of wind load, the calculation of wind-induced lateral deflection and vibration can also be important for some buildings that are required to be designed using the Dynamic Procedure. These topics, as well as vortex shedding of rounded structures, are treated separately in this Commentary.

4. The Wind Tunnel Procedure consists of tests that take into account the dynamic properties of the building structure. It can be used as an alternative to the Static and Dynamic Procedures. It is especially recommended for buildings that may be subjected to buffeting or channeling effects caused by upwind obstructions, vortex shedding, or to aerodynamic instability. It is also suitable for determining external pressure coefficients for the design of cladding on buildings whose geometry deviates markedly from common shapes. Information on wind-tunnel testing techniques can be found in ASCE/SEI 49, "Wind Tunnel Testing for Buildings and Other Structures," and in References [2] to [5].

5. Computational fluid dynamics (CFD) techniques employing turbulence closure methods, such as Reynolds Averaged Navier Stokes (RANS) modelling and Large Eddy Simulation (LES), are occasionally used during the preliminary design stage to determine approximate wind flow patterns around buildings in consideration of pedestrian comfort or pollutant dispersion. However, CFD techniques deliver results that are not sufficiently accurate or reliable for the determination of wind loads affecting structural integrity in the context of the highly complex turbulent flows around buildings. Furthermore, no accepted consensus standards currently exist that define appropriate CFD procedures (e.g., turbulence closure method, resolution of the computational grid, time step requirements, simulation length, number of wind directions to simulate, modelling of surroundings, modelling of upwind terrain). As such, the NBC does not permit CFD to be used independently of the Wind Tunnel Procedure.

6. The applicable exposure factors and some gust effect factors for the Static Procedure are specified in NBC Sentences 4.1.7.3.(5), (7), (8) and (10). All factors and coefficients for the Dynamic Procedure are given in NBC Article 4.1.7.8. Figure I-1 shows the procedure determination flow chart and provides references to applicable provisions in NBC Subsection 4.1.7. and this Commentary to help users determine wind load and effects for buildings.

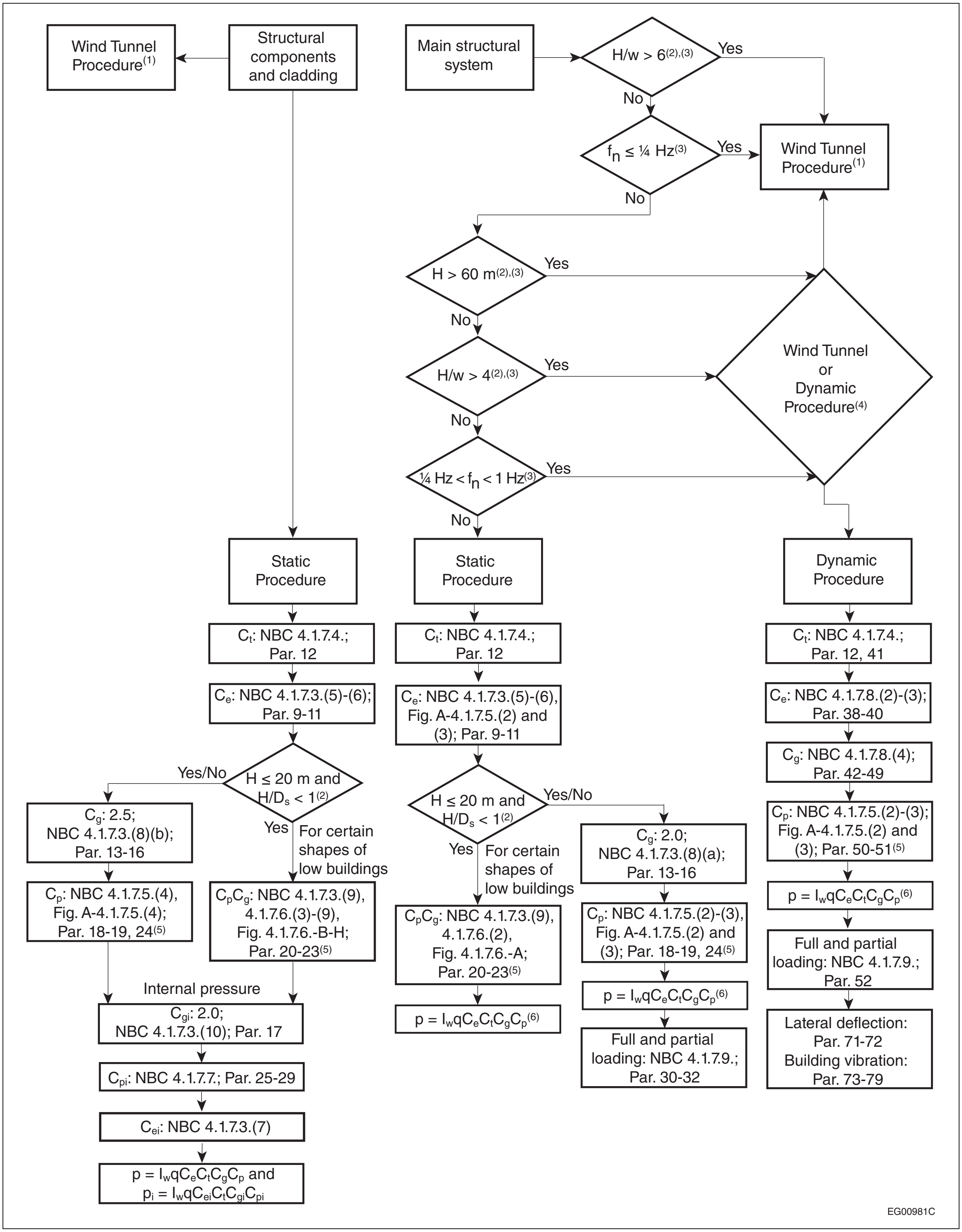


Figure I-1
Flow chart for calculating wind load and effects on buildings

Notes to Figure I-1:

(1) The Wind Tunnel Procedure is an acceptable compliance method for all cases.

(2) H is the height, D_s the smaller plan dimension, and w the effective width of the building as defined in NBC Article 4.1.7.2.
(3) See also NBC Article 4.1.7.2.
(4) The Wind Tunnel Procedure is recommended for some cases—see Paragraph 4.
(5) For round buildings and spherical or curved roofs, see Figures I-13 to I-16.
(6) The internal pressure should be considered where it could affect load on the building structure (e.g., roof uplift affecting axial load on columns).

Reference Wind Pressure

7. NBC Appendix C of Division B contains a description of the procedures followed to obtain the reference wind velocity pressures, q, and a table listing the values of q for many Canadian locations along with other climatic design data. The values of q were calculated from the annual maxima of 60 minutes moving average wind speed at a height of 10 m, $\bar{V}$, in open flat terrain and have an annual probability of being exceeded of 1 in 50 (commonly referred to as "a return period of 50 years"). This Appendix also provides information on the conversion of q to $\bar{V}$, which is needed to calculate the mean wind speed at the top of the structure, V_H (referenced in NBC Sentence 4.1.7.8.(4)).

Static Procedure

Application

8. The Static Procedure can be used to calculate the wind loads on all buildings except those identified by one of the criteria stated in NBC Sentences 4.1.7.2.(2) and (3) and in Figure I-1.

Exposure Factor, C_e

9. The exposure factor, C_e, reflects changes in wind speed with height, as well as the effects of variations in the roughness of surrounding terrain.

10. The value of C_e to be used with the Static Procedure is given in NBC Sentence 4.1.7.3.(5). It is based on the profile (variation with height) of wind-gust pressure on two types of surrounding terrain—open and rough—which are illustrated in Figures I-2 to I-5. For open terrain, the profile is assumed to obey the 0.2 power law, which is equivalent to the 0.1 power law for wind-gust speeds. For rough terrain, the 0.3 power law is assumed for the wind-gust pressure profile (equivalent to the 0.15 power law for wind-gust speed). The wind gust referred to lasts about 3 to 5 s and represents a parcel of wind, which is assumed to have an effect over the whole structure of most ordinary buildings.

Figure I-2
Example of open terrain under the Static Procedure and of Exposure A under the Dynamic Procedure for determining the exposure factor, C_e. (See also Figure I-3.) (Reproduced with the permission of the National Capital Commission ©NCC/CCN)

Figure I-3
Example of open and rough terrains under the Static Procedure. Buildings located in the foreground near the road should be designed for open-terrain exposure. Buildings that are located away from the road and deeper into the built-up area should be designed for either an intermediate exposure as given in Paragraph 11, or a rough-terrain exposure as given in Paragraph 10, depending on the distance from the road. (See also Figure I-4.) (Reproduced with the permission of the National Capital Commission ©NCC/CCN)

Figure I-4
Example of rough terrain under the Static Procedure and of Exposure B under the Dynamic Procedure. Buildings located on the periphery of the lake and open area in the background may be required to be designed for an open-terrain exposure. (Reproduced with the permission of the National Capital Commission ©NCC/CCN)

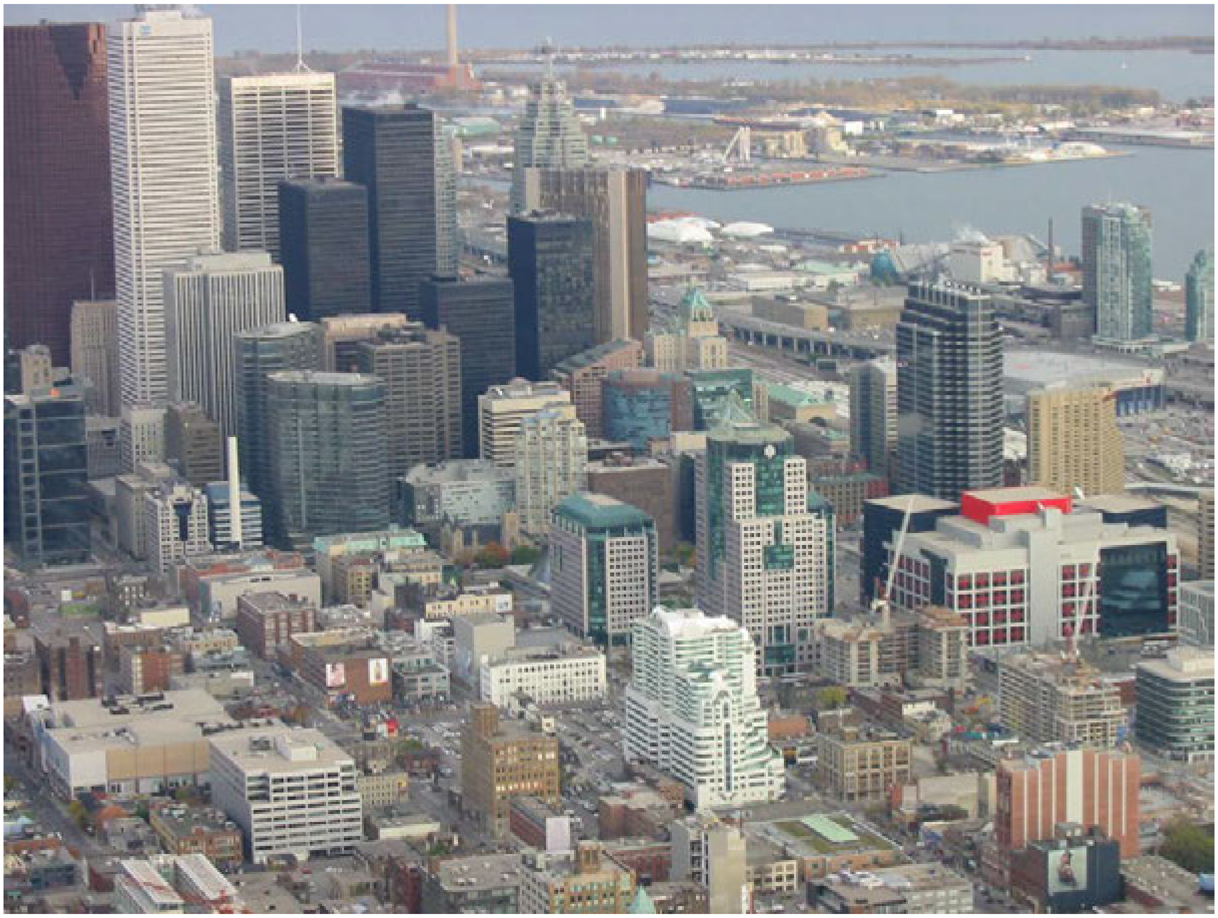


Figure I-5
Example of Exposure B under the Dynamic Procedure. Buildings located on the periphery of the lake in the right background may be required to be designed for Exposure A. In addition, tall buildings in the foreground may be required to be designed by experimental methods to account for channeling, buffeting and vortex-shedding effects. (Reproduced with the permission of The Helicopter Company, Toronto, 2003)

Changes in Terrain

11. The value of C_e for rough terrain given in NBC Sentence 4.1.7.3.(5) can be used where the rough terrain extends in the upwind direction for at least 1 km (i.e., $x_r \geq 1$ km) or 20 times the building height, H, whichever is greater. When the rough terrain extends for less than 1 km (i.e., $x_r < 1$ km) and the building is less than 50 m tall, the value of C_e may be interpolated between the values for the open and rough terrains as follows:

for x_r greater than 0.05 km and less than 1 km,

$$C_e = C_{er}\left(0.816 + 0.184\log_{10}\left(\frac{10}{x_r - 0.05}\right)\right) \leq C_{eo} \tag{1}$$

for x_r less than or equal to 0.05 km,

$$C_e = C_{eo} \tag{2}$$

where

x_r = upwind extent of rough terrain,
C_{er} = C_e for rough terrain, and
C_{eo} = C_e for open terrain.

Equations (1) and (2) are based on the studies described in Reference [6].

Speed-up over Hills and Escarpments – Topographic Factor, C_t

12. Hills and escarpments can significantly increase wind speeds near the ground. This effect is reflected by applying the topographic factor when determining the wind loads for buildings located on a hill or escarpment. The method to be used with both the Static and Dynamic Procedures is presented in NBC Article 4.1.7.4.

Hills and escarpments with slopes less than 1 in 10 are unlikely to produce significant speed-up of the wind. A more detailed discussion of this issue and other simplified models for three-dimensional hills are given in Reference [7]. Background material can be found in References [8] and [9]. Wind tunnel tests can be used to obtain design information in other cases.

Gust Effect Factors, C_g and C_{gi}

General

13. NBC Sentences 4.1.7.3.(8) and (10) contain requirements for determining the external and internal gust effect factors. These two factors, denoted by C_g and C_{gi} respectively, are defined as the ratio of the maximum effect of the loading to the mean effect of the loading. They take into account:
(a) random fluctuating wind forces caused by turbulence in the approaching wind and acting for short durations over all or part of the structure,
(b) fluctuating forces induced by the wake of the structure itself,
(c) additional inertial forces arising from motion of the structure itself as it responds to the fluctuating wind forces, and
(d) additional aerodynamic forces due to alterations in the airflow around the structure caused by its motions (aero-elastic effects).

14. All structures are affected to some degree by these fluctuating forces. The total response can be considered as a superposition of a “background component,” which acts quasi-statically, and a “resonant component,” which is due to inertial forces arising from excitation close to a natural vibration frequency. For the majority of structures, the resonant component is small and the dynamic effect can be treated by considering only the background component using normal static methods. These structures are amenable to the Static Procedure. In structures that are particularly tall, long, slender, lightweight, flexible or lightly damped, the resonant component may be dominant and so the Static Procedure cannot be used.

External Gust Effect Factor, C_g

15. The values of the external gust effect factor, C_g, for small and low-rise structures, or structures and components having a relatively high rigidity, are given in NBC Sentence 4.1.7.3.(8).

16. The peak pressure coefficients of certain low-rise structures can be determined directly from wind-tunnel tests. These coefficients are composite values of C_pC_g, incorporating the gust effect in addition to aerodynamic shape factors, and are given in NBC Article 4.1.7.6. dealing with pressure coefficients. Therefore, a gust effect factor should not be used in conjunction with these coefficients.

Internal Gust Effect Factor, C_{gi}

17. As stipulated in NBC Sentence 4.1.7.3.(10), the default value of the internal gust effect factor, C_{gi}, should be taken as 2.0. However, for large structures enclosing a single unpartitioned volume, the internal pressure takes significant time to respond to changes in external pressure, thus reducing the gust effect factor. In such cases, C_{gi} may be altered using the equation in NBC Sentence 4.1.7.3.(10). The following method of estimating C_{gi}, which accounts for the flexibility of the building envelope, can also be used:

$$C_{gi} = 1 + \frac{1}{\sqrt{1+\tau}} \qquad (3)$$

where τ is a parameter associated with the time it takes for the internal pressure to respond to changes in external pressure at openings, calculated as follows:

$$\tau = \frac{V_0}{6950A}\left[1 + 1.42 \times 10^5 \frac{A_s}{V_0}\delta\right] \tag{4}$$

where
V_0 = internal volume, in m³,
A = total area of all exterior openings of the volume, in m²,
A_s = total interior surface area of the volume (excluding slabs on grade), in m², and
δ = a measure of the flexibility of the building envelope and is the average outward deflection of the volume's envelope per unit increase in internal pressure, in m³/N.

The equation for C_{gi} provided in NBC Sentence 4.1.7.3.(10) uses a conservative approximation of τ by assuming δ is equal to zero.

The value of δ will depend on many factors, such as building size, type of cladding system, and stiffness of the supporting structure. A typical value of δ for buildings with sheet metal cladding is about 5×10^{-5} m³/N.

Example: Suppose a building's plan dimensions are 100 m × 50 m and it is 20 m high. It contains a single undivided volume, has a single opening of 5 m², and $\delta = 5 \times 10^{-5}$ m³/N. Then V_0 = 100 000 m³, A = 5 m², and A_s = 6000 + 5000 = 11 000 m². Hence

$$\begin{aligned}\tau &= \frac{10^5}{6950 \times 5}\left[1 + 1.42 \times 10^5 \frac{1.1 \times 10^4}{10^5} 5 \times 10^{-5}\right] \\ &= 2.88\,[1 + 0.78] \\ &= 5.1\end{aligned}$$

and

$$\begin{aligned}C_{gi} &= 1 + \frac{1}{\sqrt{1 + 5.1}} \\ &= 1.40\end{aligned}$$

The value of C_{gi} for this particular example would have been 1.51 if the cladding's flexibility had been ignored, i.e., if the equation in Sentence 4.1.7.3.(10) had been used.

Pressure Coefficients, C_p and C_{pi}

General

18. Pressure coefficients are the non-dimensional ratios of actual wind-induced pressures on a building surface to the velocity pressure of the wind at the reference height. They account for the effects of aerodynamic shape of the building, orientation of the surface with respect to the wind flow, and profile of the wind velocity. Pressure coefficients are usually determined from wind-tunnel experiments on small-scale models, although measurements are occasionally made on full-scale buildings. It is very important to simulate the natural velocity profile and turbulence in the wind tunnel; experiments in smooth uniform flow can be misleading.[10][11]

Directionality

19. At any geographical location, winds are the strongest for certain directions. The probability is less than 100% that the direction of the strongest wind will align with the direction that produces the highest pressure on a given surface. Therefore, the actual wind load on a given surface will be less than computed by combining the reference wind velocity pressure for the location with the peak pressure coefficient for the surface. An allowance for directionality effects has been included in the factored loads, and so no further reduction should be made to them unless the loads are determined through a detailed wind tunnel study.

External Pressure Coefficients for Buildings whose Height Is ≤ 20 m and Less than the Smaller Plan Dimension

20. Recommended external pressure coefficients for designing buildings that are no more than 20 m high are given in NBC Article 4.1.7.6. They are based on data obtained from systematic boundary-layer wind tunnel studies. In several instances, these data have been verified against full-scale measurements. The coefficients are based on the gust pressures and, consequently, include an allowance for the gust effect factor, C_g; they therefore represent the product C_pC_g. These coefficients apply to the tributary area associated with the particular element or member over which the wind pressure is assumed to act.

21. The external pressure-gust coefficients given in NBC Article 4.1.7.6. are most appropriate for buildings with a height-to-width ratio less than 1.0 and a reference height less than or equal to 20 m, where the width is based on the smaller plan dimension, D_s. Beyond these limits, NBC Article 4.1.7.5. must be used. These coefficients are based on References [12] and [13].

22. NBC Figure 4.1.7.6.-A presents values of C_pC_g for the main structural system of the building affected by wind pressures acting simultaneously on all surfaces, such as in frame buildings. The simplified load distributions shown in this Figure were developed to represent as closely as possible the structural actions (horizontal thrust, uplift and frame moments) determined directly from experiment. These results make allowance for the partial loading of gusts referred to in NBC Sentence 4.1.7.9.(1).

23. NBC Figures 4.1.7.6.-B to 4.1.7.6.-H are intended to be used for the design of cladding and structural components, such as purlins and girts, that are influenced mainly by wind acting over single surfaces. They should also be used for the design of structural elements with single surfaces, such as roofs for which moment connections are not provided at the roof/wall intersection. In this case, the edge region loads need not be included around the entire perimeter of the roof, but only adjacent to the windward edges. For roof slopes exceeding 7° where edge regions are also specified along the ridge, these increased loads need only be included on the downstream side. The loads on other edge regions can be reverted to the values specified for the interior regions.

See References [14] to [25] for additional information and clarification on the notes to NBC Figures 4.1.7.6.-B to 4.1.7.6.-E as follows:

- References [14] and [15] for facades with vertical ribs deeper than 1 m referred to in Note (5) of NBC Figure 4.1.7.6.-B and NBC Clause 4.1.7.5.(4)(b)
- References [16] and [17] for overhung roofs referred to in Note (1) of NBC Figure 4.1.7.6.-C
- Reference [18] for tributary areas larger than 100 m² referred to in Note (6) of NBC Figure 4.1.7.6.-C
- References [19] and [20] for roofs having a perimeter parapet that is 1 m high or greater referred to in Note (7) of NBC Figure 4.1.7.6.-C
- References [21] and [22] for stepped roofs referred to in Note (1) of NBC Figure 4.1.7.6.-D
- References [23] and [24] for overhung roofs referred to in Note (1) of NBC Figure 4.1.7.6.-E
- Reference [25] for hipped roofs referred to in Note (5) of NBC Figure 4.1.7.6.-E

External Pressure Coefficients for Buildings of Any Height

24. NBC Article 4.1.7.5. contains the external pressure coefficients, C_p, that can be used for the design of buildings of any height.

In NBC Sentences 4.1.7.5.(2) and (3), the coefficients are given as either time- or spatially-averaged pressure coefficients, C_p, for design of the main structural system and roofs.

NBC Sentence 4.1.7.5.(4) provides time-averaged local pressure values of C_p for the design of the cladding and secondary structural elements supporting the cladding.

NBC Sentence 4.1.7.5.(5) provides time-averaged local pressure coefficients for the design of balcony guards.

NBC Figure A-4.1.7.5.(2) and (3) presents values of C_e and C_p as defined in NBC Articles 4.1.7.3. and 4.1.7.5., respectively, for the design of the main structural system. NBC Figure A-4.1.7.5.(4)

presents values of C_p for the design of the cladding and secondary structural elements supporting the cladding.

Table I-1 indicates which NBC Figure to consult to derive pressure coefficients for all types of buildings.

Table I-1
Index of NBC Figures Containing External Pressure Coefficients

Property of Building	Structural Element	Roof Slope (α)	NBC Figure Number	Coefficient Given
$H \leq 20$ m and H/D_s(1) < 1	Main structural system	—	4.1.7.6.-A	C_pC_g
	Structural components and wall cladding	—	4.1.7.6.-B	
	Structural components and roof cladding			
	(a) general	$\alpha \leq 7°$	4.1.7.6.-C	
	(b) stepped flat	$\alpha = 0°$	4.1.7.6.-D	
	(c) gable and hipped, single-ridge	$\alpha \leq 7°$	4.1.7.6.-C	
		$\alpha > 7°$	4.1.7.6.-E	
	(d) gable, multiple-ridge	$\alpha \leq 10°$	4.1.7.6.-C	
		$\alpha > 10°$	4.1.7.6.-F[26][27]	
	(e) monosloped	$\alpha \leq 3°$	4.1.7.6.-C	
		$3° < \alpha \leq 30°$	4.1.7.6.-G[28][29]	
	(f) sawtoothed	$\alpha \leq 10°$	4.1.7.6.-C	
		$\alpha > 10°$	4.1.7.6.-H[29]	
Building of any height	Main structural system	—	A-4.1.7.5.(2) and (3)	C_p
	Structural components and roof and wall cladding	—	A-4.1.7.5.(4)	

(1) D_s = smaller plan dimension

Internal Pressure Coefficient, C_{pi}

25. The internal pressure coefficient, C_{pi}, defines the effect of wind on the air pressure inside the building and is important in the design of both cladding elements and the main structural system. The magnitude of this coefficient depends on the distribution and size of the leakage paths and openings that vent the internal air space to the exterior. With very small and uniformly distributed cracks and pores, the leakage is slow. Although the internal pressure will approximately equilibrate to the average external pressure over the exposed surface, the influence of gusts will be attenuated. If the openings are larger and more significant—on the scale of doors or windows—the internal pressure will move closer to that prevailing externally at the largest dominant opening and gust pressures will be felt within the interior.

26. Because of the changeability and uncertainty of the size and distribution of openings, internal pressure coefficients can be wide-ranging. In the face of these uncertainties, it is adequate to use the coefficients provided in NBC Table 4.1.7.7. for both the Static and Dynamic Procedures. The coefficient depends on whether there are significant openings and whether small openings producing background leakage are uniformly distributed. In this context, a large or significant opening means a single opening or a combination of openings on any one wall that offers a passage to the wind and whose area exceeds by a factor of 2 or more the leakage area of the remaining building surfaces, including the roof. Such a significant opening may be provided by main doors, shipping doors, windows and ventilators if they are open during a storm, either through expected usage or through damage.

To handle the range of circumstances that may prevail, three basic building opening categories are provided in NBC Article 4.1.7.7.

Building Opening Category 1: C_{pi} = −0.15 to 0.0

This category deals with buildings without any large or significant openings, but having small uniformly distributed openings amounting to less than 0.1% of total surface area. The value of C_{pi} should be −0.15, except where such openings alleviate an external load, in which case C_{pi} = 0 should be used. Even within buildings having small distributed openings, the internal pressure fluctuates, occasionally reaching 0. Such buildings include high-rise buildings that are nominally sealed, have no operable windows and screen doors, and are mechanically ventilated. Some less common low-rise buildings, such as windowless warehouses with door systems not prone to storm damage, also fall into this category.

Building Opening Category 2: C_{pi} = −0.45 to +0.3

This category covers buildings in which significant openings, if there are any, can be relied on to be closed during storms but in which background leakage may not be uniformly distributed. Most low-rise buildings fall into this category provided that all elements—especially shipping doors—are designed to be fully wind-resistant. Most high-rise buildings with operable windows or balcony doors also fall into this category.

Building Opening Category 3: C_{pi} = −0.7 to +0.7

This category covers buildings with large or significant openings through which gusts are transmitted to the interior. Examples of such buildings include those with a permanently open door or window and sheds with one or more open sides. Post-disaster buildings should also be treated as Category 3 in order to achieve a higher level of reliability.

27. An ever-present threat in severe storms is the breakage of large unprotected glass areas and other vulnerable components by flying debris. Structures required in post-disaster services should be capable of withstanding all the consequences of failure of glass and conform to the requirements of Building Opening Category 3. For other structures in which the glass is designed for wind and there is adequate protection against roof uplift, the contingency of glass damage due to debris is covered by normal load factors for wind.

28. In most cases, there is no need to consider non-uniform internal pressures except in the design of internal partitions (see NBC Sentence 4.1.7.10.(1)). Thus, for most structural design, the two limiting values of internal pressure can be considered separately unless interior compartments of the building are well sealed and wind damage or the like could expose one area of the building to Building Opening Category 3 conditions while the rest of the building remains in Building Opening Category 1 or 2, resulting in unbalanced internal pressures.

29. Internal pressures are also affected by mechanical ventilation systems and by the stack effect due to different inside and outside air temperatures. Under normal operation, mechanical ventilation systems create a differential of less than 0.1 kPa across walls, but the stack effect due to differences in temperature of 40°C could amount to a differential of 0.2 kPa per 100 m of building height.[30]

Partial Loading

30. Partial wind loading can, in some cases, cause more severe effects than full loading. Pressure patterns observed in turbulent wind indicate reduced loading on portions of the building faces, which can produce additional torsion due to horizontal shifting of the wind-load vector. Reduced but simultaneous loading along both major axes can be induced by wind blowing diagonally to the building, which can produce higher stresses in some structural members than by wind blowing along any one major axis. Other structures, such as curved roofs, may undergo larger stresses under partial loading. NBC Sentence 4.1.7.9.(1) therefore requires all buildings to be designed for partial loading as well as full loading.

31. Low buildings designed by the Static Procedure to the specifications of NBC Figure 4.1.7.6.-A do not need to have further unbalanced loads (see Paragraph 22). Taller buildings, in addition to being designed for the full wind load along each of the principal axes, as shown in Case A of NBC Figure A-4.1.7.9.(1), should be checked for maximum additional torsion arising from partial loadings created by applying the wind pressure to only a part of the building face areas, as shown in Case B of NBC Figure A-4.1.7.9.(1).

32. To account for the potentially more severe effects induced by diagonal wind, and also for the tendency of structures to sway in the across-wind direction, taller structures should be designed to resist 75% of the maximum wind pressures for each of the principal directions applied simultaneously, as shown in Case C of NBC Figure A-4.1.7.9.(1). In addition, the influence of removing 50% of the Case C loads from parts of the face areas, as shown in Case D of NBC Figure A-4.1.7.9.(1), which maximizes torsion, should be investigated. Further discussion on combined loading effects can be found in References [31] and [32].

Dynamic Procedure

Application

33. NBC Sentence 4.1.7.1.(3) requires the use of the Dynamic or Wind Tunnel Procedure for buildings whose height is greater than 4 times their minimum effective width, or greater than 60 m, or other buildings whose lowest natural frequency, f_n, is < 1 Hz and > ¼ Hz as determined by rational analysis. Minimum effective width is defined in NBC Sentence 4.1.7.2.(2).

34. NBC Sentence 4.1.7.1.(4) requires that the Wind Tunnel Procedure be used for buildings whose lowest natural frequency, f_n, is ≤ ¼ Hz as determined by rational analysis or whose height is more than 6 times their minimum effective width.

35. The lowest natural frequency, f_n, referred to in NBC Sentences 4.1.7.2.(2) and (3) may be determined using Finite Element Modelling or estimated using the following approach (Rayleigh's method):
- the building is divided into a number, N, of vertical levels, each level typically being one floor denoted as the i^{th} level or the roof;
- each level or floor has an associated wind force, F_i, which may be computed using the Static Procedure;
- each level or floor also has an associated mass, M_i; and
- the horizontal deflections of each floor, x_i, caused by F_i are computed using appropriate structural static analysis methods, including the deflection of the top level, x_N, i.e., at the N^{th} level.

The lowest natural frequency, in Hz, can then be estimated using the following equation:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{\sum_{i=1}^{N} F_i \frac{x_i}{x_N}}{x_N \sum_{i=1}^{N} M_i \left(\frac{x_i}{x_N}\right)^2}} \qquad (5)$$

It is important to note that the frequency determined from Equation (5) is to be used only for estimating the lowest natural frequency that triggers the requirement to use the Dynamic Procedure for design for wind forces and should not be used in the calculation of design seismic loads.

36. In the Dynamic Procedure for calculating wind load on the building structure, the exposure factor, C_e, and external gust effect factor, C_g, are different from the factors used in the Static Procedure, but the pressure coefficient, C_p, is the same. See Figure I-1 for guidance on how the Dynamic Procedure for the structure is carried out in conjunction with the Static Procedure for the cladding.

37. In addition to the calculation of wind load, the calculation of wind-induced lateral deflection, vibration and vortex-shedding effect can also be important for some buildings that are required to be treated by the Dynamic Procedure. These topics are dealt with separately under the sections of this Commentary entitled Lateral Deflection of Tall Buildings, Building Vibration and Vortex Shedding.

Exposure Factor, C_e

38. In the Dynamic Procedure, the exposure factor, C_e, is based on the profile of mean wind speed, which varies considerably with the general roughness of the terrain over which the wind has been blowing before it reaches the building. To determine the exposure factor, two categories of terrain exposure have been established and are illustrated in Figures I-2 to I-5.

39. Exposure B should not be used unless the applicable terrain roughness persists in the upwind direction for at least 1.0 km or 20 times the height of the building, H, whichever is larger, and the exposure factor should be recalculated if the roughness of the terrain differs from one direction to another.

40. In addition to being used to calculate pressures on building surfaces, the exposure factor is needed for calculating the hourly mean wind speed at the top of the building, V_H, and the gust effect factor, C_g (NBC Sentence 4.1.7.8.(4)).

Speed-up over Hills and Escarpments

41. The topographic factor, C_t, covered in NBC Article 4.1.7.4., is used to account for speed-up over hills and escarpments in both the Static and Dynamic Procedures. Speed-up, which principally affects the mean wind speed and not the amplitude of the turbulent fluctuations, is accounted for in the formulation of C_t.

Gust Effect Factor, C_g

General

42. The general discussion on the gust effect factor presented in Paragraphs 13 and 14 under the Static Procedure is also applicable to the Dynamic Procedure.

External Gust Effect Factor, C_g

43. The critical damping ratio, β, which is needed as input in NBC Article 4.1.7.8. to calculate wind response, is based mainly on experiments on real structures. Expressed as a fraction of critical damping, values commonly used in the design of buildings with steel frames and concrete frames are 0.01 and 0.02, respectively. Examples of composite buildings to which a β value of 0.015 applies include buildings with steel framing and concrete cores and buildings where both steel and concrete resist lateral loads. On the other hand, masts, stacks and extremely slender buildings, which resist wind load primarily through cantilever action, may have much lower inherent or structural damping. Aerodynamic damping in the along-wind direction becomes significant at high wind speeds, but plays no useful role in limiting cross-wind motion induced by vortex shedding. Spread footings on soft or medium-stiff soil provide higher damping compared to pile foundations or spread footings on stiff soil and rock. Damping values measured from more than 20 stacks are tabulated in Reference [33] and the results from 5 more stacks are given in Reference [34]. The logarithmic decrement mentioned therein is 2π times the critical damping ratio. Sachs[33] concludes by stating a range of 0.002 to 0.008 for β for the total damping of closed circular and unlined welded steel stacks, and suggests that the minimum value be used in design. Corresponding ranges for lined welded steel stacks and for unlined reinforced concrete stacks are given as 0.005 to 0.01 and 0.01 to 0.02, respectively.

Explanatory Notes Regarding σ/μ and g_p

44. The response of a tall, slender building to a randomly fluctuating force can be evaluated rather simply by treating it as a rigid, spring-mounted cantilever whose dynamic properties are specified by a single natural frequency and an appropriate damping value. The variance of the output quantity or loading effect is the area under the spectrum of the input quantity (the forcing function) after it has been multiplied by the transfer function. The transfer function is the square of the well-known dynamic load magnification factor for a one-degree-of-freedom oscillating mechanical system.

45. In the case of wind as the randomly fluctuating force, the spectrum of the wind speed must first be multiplied by another transfer function called the aerodynamic admittance function, which in effect describes how the turbulence in the wind is modified by its encounter with the building, at least insofar as its ability to produce a loading effect on the structure is concerned.

46. For the purpose of calculating the ratio of the root-mean-square loading effect, σ, to the mean loading effect, μ—i.e., the coefficient of variation, σ/μ—the spectrum of the wind speed is represented by an algebraic expression derived from observations of real wind. The aerodynamic admittance function is also an algebraic expression, computed on the basis of somewhat simplified assumptions but appearing to be in reasonable agreement with experimental evidence. The spectrum of wind speed is a function of frequency having the shape of a rather broad hump (see NBC Figure A-4.1.7.8.(4)-C). The effect of the aerodynamic admittance is to reduce the ordinates of the curve to the right of the hump more and more as the frequency increases. This is partly a reflection of the reduced effectiveness of small gusts in loading a large area. The effect of the dynamic load magnification factor or mechanical admittance is to create a new peak or hump centred at the natural frequency of the structure—usually well to the right of the broad peak—which represents the maximum density of fluctuating force of the wind.

47. The area under the loading effect spectrum, the square root of which is the coefficient of variation, σ/μ, is taken as the sum of two components: the area under the broad hump, which must be integrated numerically for each structure, and the area under the resonance peak, for which a single analytic expression is available. These components are represented by B and sF/β, respectively, in the expression for σ/μ in NBC Sentence 4.1.7.8.(4). The factor K/C_{eH} can be thought of as scaling the result for the appropriate input turbulence level. If resonance effects are small, then sF/β will be small compared to the background turbulence, B, and vice versa.

48. The peak factor, g_p, depends on the average number of times the mean value of the loading effect is surpassed during the averaging time of 1 hour (3600 s). The functional relationship in NBC Figure A-4.1.7.8.(4)-A holds when the probability distribution of the loading effect is normal (Gaussian).[35]

Sample Calculation of C_g

49. The following sample calculation illustrates how to calculate the gust effect factor, C_g.

Building properties and site assumptions:
- Height, H: 183 m
- Across-wind effective width, w: 30.5 m
- Along-wind effective depth, d: 30.5 m
- Fundamental natural frequency, f_{nD}: 0.2 Hz
- Critical damping ratio, β: 0.015
- Terrain of site: Exposure B
- Reference wind speed, $\overline{V}$, at 10 m and in open terrain: 27.4 m/s

Step 1: Calculate required parameters.

C_{eH} = 1.90 (from NBC Figure A-4.1.7.8.(2) and (3))

$V_H = \overline{V}\sqrt{C_{eH}}$ (as per NBC Sentence 4.1.7.8.(4))

$= 27.4 \times \sqrt{1.90}$

$= 37.8$ m/s

w/H = aspect ratio

$= 30.5/183$

$= 0.17$

f_{nD}/V_H = wave number for calculation of F

$= 0.2/37.8$

$= 0.0053$

$f_{nD}H/V_H$ = reduced frequency for calculation of s

$= 0.2 \times 183/37.8$

$= 0.968$

Step 2: Calculate σ/μ (as per NBC Sentence 4.1.7.8.(4)) using the following parameters:
K = 0.10 for Exposure B,
B = 0.62 (from NBC Figure 4.1.7.8),
s = 0.11 (from NBC Figure A-4.1.7.8.(4)-B),
F = 0.28 (from NBC Figure A-4.1.7.8.(4)-C), and
β = 0.015 (given).

$$\begin{aligned}\sigma/\mu &= \sqrt{\frac{K}{C_{eH}}\left(B + \frac{sF}{\beta}\right)}\\ &= \sqrt{\frac{0.10}{1.90}\left(0.62 + \frac{0.11 \times 0.28}{0.015}\right)}\\ &= 0.375\end{aligned}$$

Step 3: Calculate υ (as per NBC Sentence 4.1.7.8.(4)).

$$\begin{aligned}\upsilon &= f_{nD}\sqrt{\frac{sF}{sF + \beta B}}\\ &= 0.2\sqrt{\frac{0.11 \times 0.28}{0.11 \times 0.28 + 0.015 \times 0.62}}\\ &= 0.175/s\end{aligned}$$

Step 4: Obtain peak factor, g_p.
g_p = 3.75 (from NBC Figure A-4.1.7.8.(4)-A)

Step 5: Calculate C_g (as per NBC Sentence 4.1.7.8.(4)).
C_g = 1 + $g_p(\sigma/\mu)$
= 1 + 3.75(0.375)
= 2.41

Pressure Coefficients, C_p

General

50. The general discussion presented in Paragraphs 18 and 19 under the Static Procedure is also applicable to the Dynamic Procedure.

External Pressure Coefficient, C_p

51. The coefficients given in NBC Article 4.1.7.5. under the Static Procedure are also applicable to the Dynamic Procedure (see Paragraph 24).

Partial Loading

52. Refer to Paragraphs 30 to 32 for partial loading requirements.

Wind Load on Miscellaneous Structures

Roof-Mounted Solar Arrays

53. A solar array is a collection of solar panels—typically photovoltaic modules or solar thermal panels—grouped into interconnected rows. A roof can have multiple arrays, which are typically separated by walkways, skylights and other rooftop equipment. Solar panels receive solar radiation and convert it to electricity or heat energy. In this Commentary, only flat modules and panels are considered.

54. Information on wind load calculations for solar arrays is provided in Paragraphs 56 and 57. These calculations assume the panels and their mounting system are rigid so there is no allowance for the effects of wind-induced vibrations on them. However, if the panels and their mounting system have a natural frequency less than about 10 Hz, loads could potentially be magnified due to vibrations. In such cases, expert opinion should be sought and a more detailed analysis of the dynamic effects should be carried out.

Snow Loads

55. It is important to note that the construction of solar arrays on a roof can significantly affect the distribution of snow loads: snow may pile up at the foot of the panels and the general movement of snow upon the roof may be affected, potentially increasing snow accumulation effects. Designers should be aware that the accumulation of snow around collectors can impact the calculations described in Paragraphs 56 and 57: for example, accumulated snow may obstruct the ventilation areas between the roof and the underside of the panels, which could affect pressure equalization. As such, anchorage of the arrays to the roof and of the panels to the mounting system should be designed with γ_a taken as 1 in Paragraph 56 unless it can be shown that the accumulation of snow and ice will not affect pressure equalization.

Wind Loads

56. Wind loads on solar arrays that are close to and parallel to the roof's surface tend to be lower than those on a bare roof due to pressure equalization.[36][37] This difference can be accounted for by applying a pressure equalization factor, γ_a, to the design pressure. See Figure I-6 for values of γ_a based on tributary area.

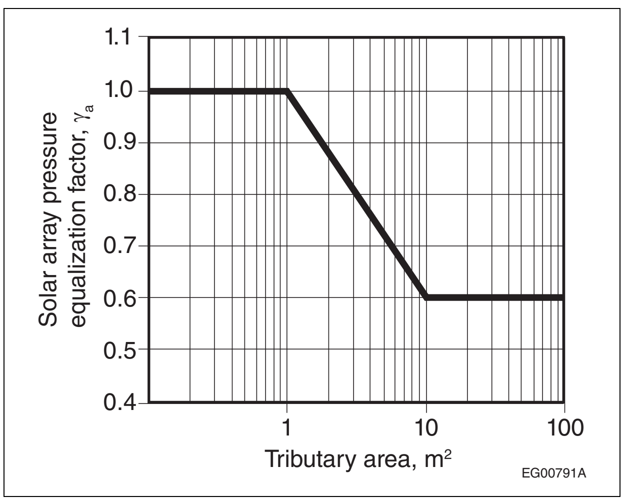


Figure I-6
Solar array pressure equalization factor, γ_a, for enclosed and partially enclosed buildings of any height

For pressure equalization to occur, limits on panel size, height and proximity must be observed. The design procedure proposed in this section is based on panels that are no more than 2 m long, installed less than 250 mm above the roof's surface with a minimum gap of 6 mm between panels at a minimum of every 2 m along a row and between each row. Installing the panels at lower heights and with larger gaps could further decrease the wind loads, but wind tunnel testing would be required to ascertain this. Arrays must be located at least $2h_2$ (h_2 being the height of the upper edge of a solar panel above the roof and h being the mean roof height) from a roof edge, a gable ridge, or a hip ridge for a value of $\gamma_a < 1.0$ to be used. Panels around the edge of an array may be affected by higher wind loads, which can be accounted for by applying an edge factor, E, as follows:

E = 1.5 for panels that are exposed and for those within a distance $1.5L_p$ (L_p being the length of the panel chord) from the end of a row at an exposed edge of the array, and

E = 1.0 for unexposed panels.

A panel is considered exposed if the distance d_1 to the roof's edge > 0.5h, and either the distance d_1 to an adjacent array or the distance d_2 to an adjacent row = max($4h_2$,1.2 m) (d_1 being the horizontal distance orthogonal to the panel edge to a panel in an adjacent array or the roof's edge (ignoring any rooftop equipment) and d_2 being the horizontal distance from the edge of one panel to the nearest edge in the next row). See Figure I-7.

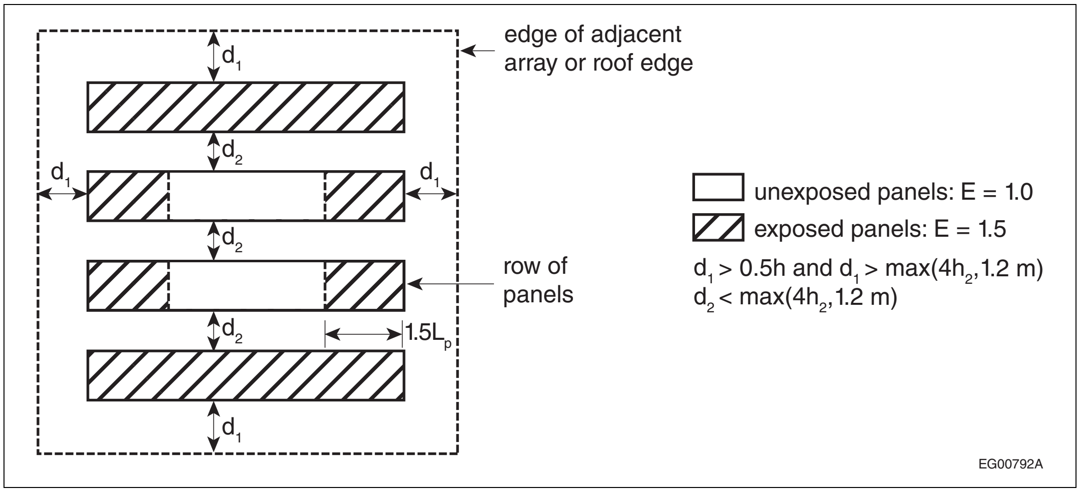


Figure I-7
Definition sketch of exposed and unexposed panels in a roof-mounted solar array

The net design wind pressure for solar panels that meet all the requirements mentioned above is calculated as follows:

$$p = I_W q C_e C_t C_g C_p E \gamma_a$$

where I_w, q, C_e, and C_t, and C_gC_p are determined for the roof cladding on the building on which the solar panels are being installed using the Static Procedure in NBC Article 4.1.7.3. Furthermore, C_gC_p is determined using the tributary area for the component under consideration.

The area of a roof assembly that is covered by an array need not be designed for the simultaneous application of the solar array wind loads and roof wind loads. The roof should also be designed for cases where the solar arrays have been removed.

57. This section provides guidance on wind load calculations for low-profile solar arrays installed on low-sloped roofs (flat roofs or gable or hip roofs with slopes less than 7°) on buildings of any height. This type of installation is common and has been subjected to significant wind-tunnel testing. The design procedure proposed in this section intentionally has a limited range of application, with h_2 limited to 1.2 m, L_p limited to 2.0 m, and h_1 (height of the lower edge of a solar panel above the roof) limited to 0.6 m, otherwise windflow under the panels could cause uplift exceeding that covered in these recommendations. Wind tunnel data (e.g., Kopp[37]) show that higher values of h_1 and h_2 or of L_p increase the wind loads. The procedure is not applicable to panels mounted on open structures because the applicable test data are from studies on enclosed structures, which have different aerodynamics than open structures.

The net design wind pressure for solar panels that conform to these limits is calculated as follows:

$$p = I_W q C_e C_t C_g C_p$$

where I_w, q, C_e, and C_t are determined for the roof cladding on the building on which the solar panels are being installed using the Static Procedure in NBC Article 4.1.7.3. and C_gC_p is calculated using Equation (6).

Parapets typically worsen the wind loads on solar panels, particularly on wider buildings; the panel parapet factor, γ_p, accounts for this effect. Large panel chords typically worsen the wind loads on arrays; the panel chord factor, γ_c, accounts for this effect. Wind loads on solar panels are strongly dependent on building size.[37][38] As such, the net pressure-gust coefficients are denoted $(C_gC_p)_n$ since the values need to be adjusted for building size. The net pressure-gust coefficient for the applicable tributary area is calculated as follows:

$$C_gC_p = \gamma_p \gamma_c E \left(C_gC_p\right)_n \tag{6}$$

where

γ_p = min(1.2,0.9 + h_{pt}/h) (h_{pt} being the height of the parapet above the roof's surface),

γ_c = max(0.6 + 0.2L_p,0.8),

E = edge factor, as defined in Paragraph 56, and

$(C_gC_p)_n$ = normalized area-averaged pressure-gust coefficients, as defined in Figure I-8.

The area of a roof assembly that is covered by an array need not be designed for the simultaneous application of the solar array wind loads and roof wind loads. The roof should also be designed for cases where the solar arrays have been removed.

Roof zones 2 and 3 shown in Figure I-8 are larger than the analogous zones for determining loads on roof cladding (see NBC Figure 4.1.7.6.-C and explanation in Kopp[37] and Banks[39]). For buildings with non-rectangular plans, the recommendations in SEAOC[38] can be used for guidance.

The $(C_gC_p)_n$ curves shown in Figure I-8 are derived from wind tunnel test data for the range of parameters allowed by the Figure. They are created based on a methodology consistent with that used for the component and cladding loads.

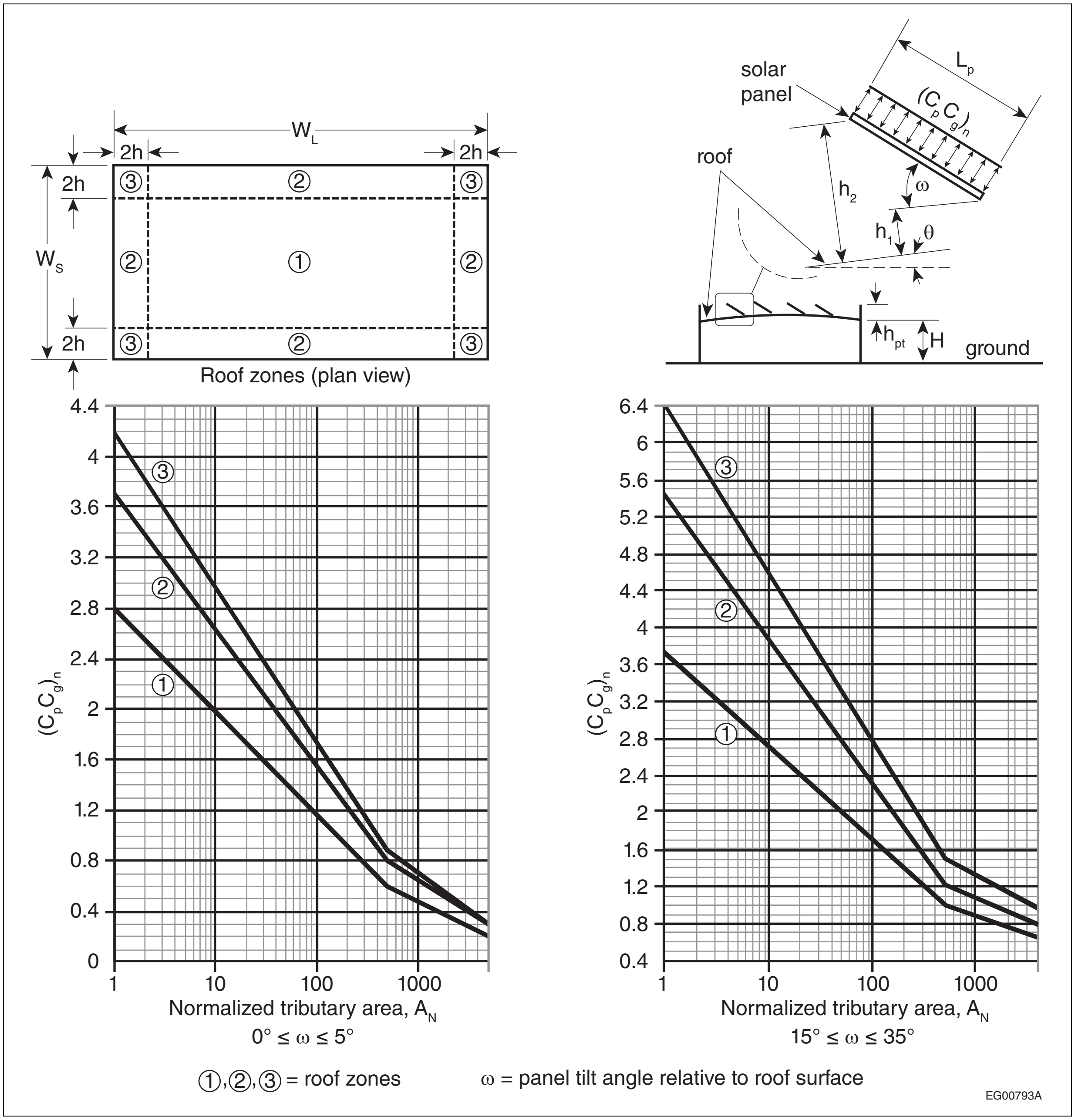


Figure I-8
Normalized area-averaged pressure-gust coefficients, $(C_gC_p)_n$, for solar arrays mounted on low-sloped roofs

Notes to Figure I-8:

(1) The $(C_gC_p)_n$ curves shown are for panels with $L_p \le 2$ m installed as follows: $h_1 \le 0.6$ m, $h_2 \le 1.2$ m, $\omega \le 35°$ and a minimum gap of 6 mm between panels at a minimum of every 2 m along a row and between each row. For panels with $\omega \le 2°$ and $h_2 \le 250$ mm, the procedure described in Paragraph 56 can be used. For panels with $5° < \omega < 15°$, linear interpolation is permitted.

(2) $(C_gC_p)_n$ values are for both positive and negative values.

(3) As per CSA SPE-900, "Solar Photovoltaic Rooftop-Installation Best Practices Guideline," the perimeter of the roof should be free of solar panels to allow roof access:
- for buildings whose smallest horizontal dimension, W_s, < 73 m, the minimum horizontal clear distance between the panels and the edge of the roof must be the larger of $2(h_2 - h_{pt})$ and 1.2 m;
- for buildings whose smallest horizontal dimension, W_s, ≥ 73 m, the minimum horizontal clear distance between the panels and the edge of the roof must be the larger of $2(h_2 - h_{pt})$ and 1.8 m.

(4) $\mathrm{A_N} = \left(\frac{1000}{[\max(\mathrm{L_b},5)]^2}\right) \times \mathrm{A}$, where

A_N = tributary area for array component under consideration, normalized for building size,

A = dimensional tributary area, in m², for the array component under consideration, and

L_b = min[0.4(h × W_L)$^{0.5}$,h,W_s], where
L_b = normalized building length,
h = mean roof height,
W_L = width of the building on its longest side, and
W_S = width of the building on its shortest side.

Sample Calculation of Net Design Wind Pressure, $p = I_W q C_e C_t C_g C_p$

This sample calculation demonstrates how to calculate net design wind pressure for solar panels located on a flat roof with no parapet on a building in Toronto. (This is not a full design example including the mounting system and anchorage loads: it serves only to clarify the calculation procedure using the tributary area of a single panel.) See Figure I-9 for the basic geometric layout of the multiple arrays on the roof.

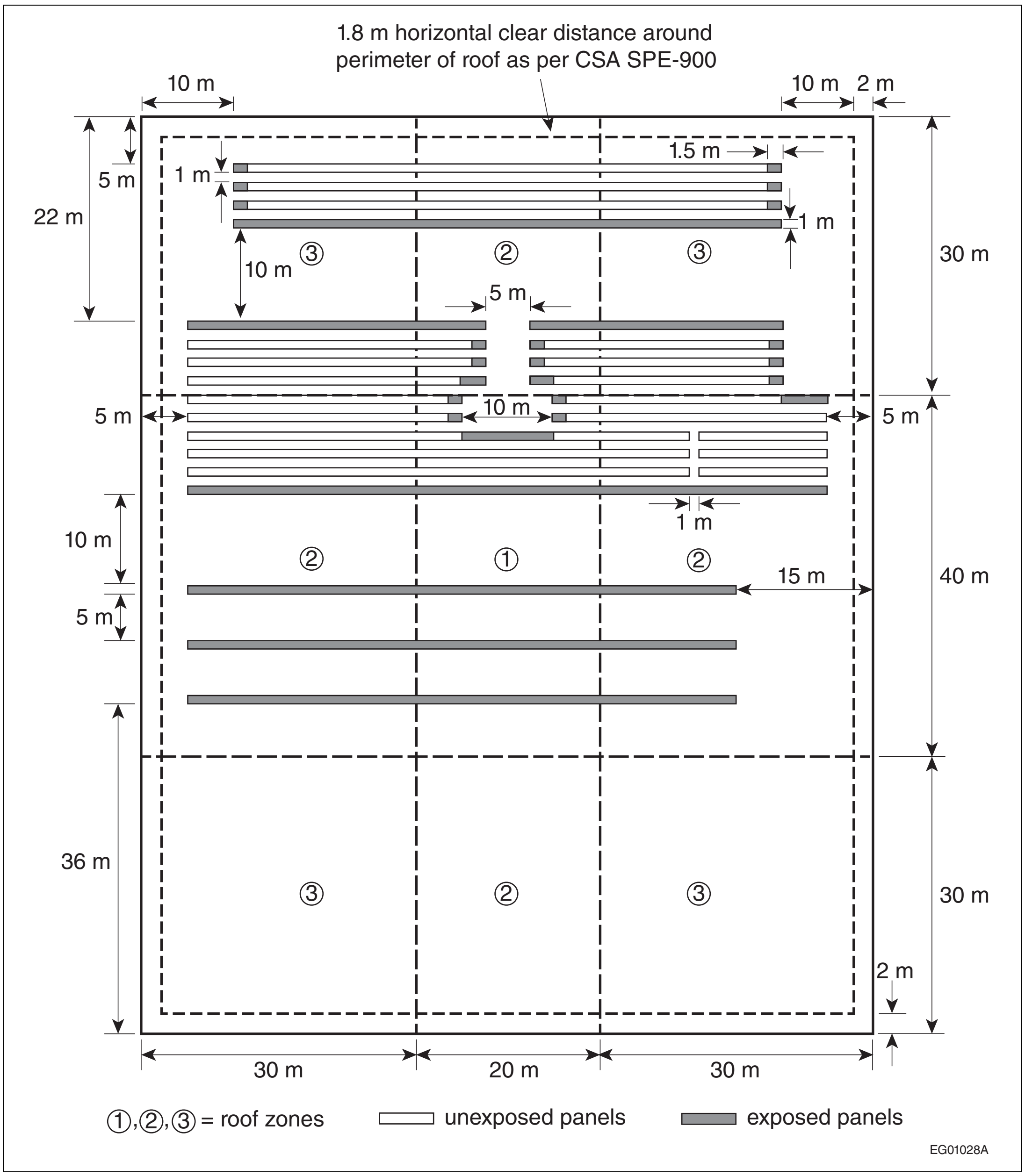


Figure I-9
Schematic of roof-mounted solar arrays (plan view) in sample calculation

Building dimensions:
reference height: h = 15 m
W_S = 80 m
W_L = 100 m

Photovoltaic panel dimensions and tilt angle:
1.0 m (chord length) by 1.8 m
h_1 = 0.5 m
h_2 = 1.0 m
ω = 30°

Step 1: Determine size of roof zones.
Zone 1: $(W_L - 4h) \times (W_S - 4h) = (100\text{ m} - 60\text{ m}) \times (80\text{ m} - 60\text{ m}) = 40\text{ m} \times 20\text{ m}$
Zone 2: $(W_S - 4h) \times 2h = (80\text{ m} - 60\text{ m}) \times 30\text{ m} = 20\text{ m} \times 30\text{ m}$
Zone 3: $2h \times 2h = 30\text{ m} \times 30\text{ m}$

Step 2: Determine the factors and reference velocity pressure (see Paragraph 57).
Importance factor (from NBC Table 4.1.7.3., Normal Importance Category): $I_W = 1.0$
Reference velocity pressure for Toronto (from NBC Table C-2): q = 0.44 kPa
Exposure factor (from NBC Clause 4.1.7.3.(5)(b)): $C_e = 0.7(h/12)^{0.3} = 0.7(15/12)^{0.3} = 0.75$
Topographic factor (from NBC Sentence 4.1.7.4.(1)): $C_t = 1.0$
Net pressure coefficient (see Paragraph 57 and Step 5): $C_gC_p = \gamma_p \gamma_c E(C_gC_p)_n$
Panel parapet factor (see Paragraph 57): $\gamma_p = \min(1.2, 0.9 + h_{pt}/h) = \min(1.2, 0.9) = 0.9$
Panel chord factor (see Paragraph 57): $\gamma_c = \max(0.6 + 0.2L_p, 0.8) = \max([0.6 + 0.2 \cdot 1.0], 0.8) = 0.8$
Edge factor: E = 1.5 for exposed panel and 1.0 for unexposed panel
$(C_gC_p)_n$ (from Figure I-8 and Steps 3 and 4)

Step 3: Calculate A_N (see notes to Figure I-8).

$$A_N = \left(\frac{1000}{[\max(L_b, 5)]^2}\right) \times A$$

where
$L_b = \min[0.4(h \times W_L)^{0.5}, h, W_S]$
$= \min[0.4(15 \times 100)^{0.5}, 15, 80]$
$= \min[24, 15, 80]$
$= 15$ m, and
A = area of the panel
$= 1.0\text{ m} \times 1.8\text{ m}$
$= 1.8\text{ m}^2$

Therefore

$$\begin{aligned} A_N &= \left(\frac{1000}{[\max(15, 5)]^2}\right) \times A \\ &= \left(\frac{1000}{15^2}\right) \times 1.8 \\ &= 8 \end{aligned}$$

Step 4: Determine $(C_gC_p)_n$ values at A_N = 8 using Figure I-8.
$(C_gC_p)_{n1} = 2.8$
$(C_gC_p)_{n2} = 4.0$
$(C_gC_p)_{n3} = 4.8$

Step 5: Calculate C_gC_p for each roof zone.
$(C_gC_p)_{1exposed} = 0.9 \times 0.8 \times 1.5 \times 2.8 = 3.0$
$(C_gC_p)_{2exposed} = 0.9 \times 0.8 \times 1.5 \times 4.0 = 4.3$

$(C_gC_p)_{3exposed}$ = 0.9 × 0.8 × 1.5 × 4.8 = 5.1

$(C_gC_p)_{1unexposed}$ = 0.9 × 0.8 × 1.0 × 2.8 = 2.0
$(C_gC_p)_{2unexposed}$ = 0.9 × 0.8 × 1.0 × 4.0 = 2.9
$(C_gC_p)_{3unexposed}$ = 0.9 × 0.8 × 1.0 × 4.8 = 3.4

Step 6: Calculate net pressure for each roof zone.

$$p = I_W q C_e C_t C_g C_p$$

$p_{1exposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 3.0 = 1.0 kPa
$p_{2exposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 4.3 = 1.4 kPa
$p_{3exposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 5.1 = 1.7 kPa

$p_{1unexposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 2.0 = 0.7 kPa
$p_{2unexposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 2.9 = 1.0 kPa
$p_{3unexposed}$ = 1.0 × 0.44 kPa × 0.75 × 1.0 × 3.4 = 1.1 kPa

Interior Walls and Partitions

58. If windows are broken during a storm, considerable pressure differences can result across interior walls and partitions in high-rise buildings, as well as in low-rise buildings in exposed locations. In certain locations, almost the full pressure difference between the windward and leeward sides of the building could be applied across interior walls or partitions. For example, when a large window in a small room on the windward side is broken by flying debris, the full positive pressure is exerted on the walls of that room. Similar conditions could prevail in an apartment building with operable windows or doors. This pressure difference could be aggravated by mechanical ventilation and winter-time stack effects in a tall building. On the other hand, experience does not indicate many failures of interior walls due to pressure differences, and thus interior walls and partitions are not required to be designed for the maximum possible pressure difference. An unfactored pressure difference of at least 0.25 kPa is suggested and a value of 0.5 kPa or higher may be appropriate in cases where the exterior wind pressures are likely to be transmitted to the interior walls and partitions through large openings in the exterior envelope.

Unenclosed Parking Structures

59. For multi-level, unenclosed parking structures, the exposed exterior area is reduced compared with enclosed structures. However, interior parts of the structure and the vehicles parked there are subject to additional wind forces not present in enclosed structures. In lieu of a detailed analysis of the specific structure under consideration, a reasonable and conservative assumption is to treat the unenclosed parking structure as though it were enclosed.

Structural Members and Frames, and Rounded Structures

60. Although the NBC deals primarily with building structures, the present Commentary has a long tradition of providing guidance on determining the wind load on other types of structures. Figures I-11 and I-13 to I-22, which are derived from SIA 160(1956),[40] "Standards for Load Assumptions, Acceptance and Inspection of Structures," provide such guidance. The Figures are based on wind-tunnel experiments in which the correct velocity profile and wind turbulence were not simulated; they should therefore be regarded with caution. Note that many of these Figures provide formulae for the total wind load rather than the wind pressure as given by the NBC, and hence use a force coefficient rather than a pressure coefficient. The exposure and gust effect factors required in the Figures to calculate the wind load can be determined by using either the Static Procedure, the Dynamic Procedure, or Vortex Shedding of rounded structures described in this Commentary, as deemed appropriate.

61. Wind loads on standalone structural members, and on frames, trusses and lattices made of such members, can be calculated using Figures I-18 to I-22. The subscript ∞ in these Figures indicates that the coefficients apply to structural members of infinite lengths. The coefficients are multiplied by a reduction factor, k, for structural members of finite lengths. If a structural member cantilevers from a large plate or wall, k should be calculated for a slenderness based on twice the actual length. If a member terminates with both ends in large plates or walls, the reduction factors for infinite length should be used.

62. For framing members that are located behind each other in the direction of the wind, the shielding effect may be taken into account. The shielded parts of the leeward members should be designed with the reduced pressure, q_x, according to Figure I-20. A detailed discussion of the loads on unclad building frameworks is given in Reference [41].

63. As the shape of a structure may change during erection, the wind loads may be temporarily more critical during erection than after completion of the structure.[23] These increased wind loads should be taken into account using the appropriate coefficients from NBC Figures 4.1.7.6.-A to 4.1.7.6.-H and A-4.1.7.5.(2) and (3), and Figures I-11 to I-22.

64. For constructions made of circular sections with $D\sqrt{(qC_tC_e)} < 0.167$ and $A_s/A > 0.3$, the shielding factors can be taken by approximation from Figure I-17. If $D\sqrt{(qC_tC_e)} \geq 0.167$, the shielding effect is small, and for a solidity ratio, A_s/A, ≤ 0.3, it can be taken into account by a constant shielding factor, k_x, = 0.95.

65. For rounded structures (in contrast to sharp-edged structures), the cross-wind pressures vary with the wind velocity and depend strongly on the Reynolds Number. Pressure coefficients for some rounded structures are given in Figures I-13, I-14, I-17 and I-22, in which the Reynolds Number is expressed differently from the conventional one, by $D\sqrt{(qC_tC_e)}$, where D is the diameter of the sphere or cylinder in m and q is the velocity pressure in kPa. To convert to the conventional Reynolds Number, multiply $D\sqrt{(qC_tC_e)}$ by 2.7×10^6.

66. The roughness of rounded structures may be of considerable importance. With reference to Figure I-13, metal, concrete, timber and well-laid masonry without parging can be considered as having a “moderately smooth” surface. Surfaces with ribs projecting more than 2% of the diameter are considered “very rough.” In case of doubt, coefficients that result in the greater forces should be used. For cylindrical and spherical objects with substantial stiffening ribs, supports and attached structural members, the pressure coefficients depend on the type, location and relative magnitude of these roughnesses. For vortex shedding of circular cylinders, see Reference [42].

Increased Wind Load Due to Icing

67. In locations where the strongest winds and icing may occur simultaneously, forces on structural members, cables and ropes must be calculated assuming an ice covering based on climate and local experience. For the iced condition, values of C_f given in Figure I-17 for thick wire cables for a “rough” surface should be used. Information on icing loads can be obtained from CSA S37, “Antennas, Towers, and Antenna-Supporting Structures,” and ISO 12494, “Atmospheric Icing of Structures.”

Design Basis for Glass (NBC Article 4.3.6.1.)

68. In previous editions of the NBC, CAN/CGSB-12.20-M, “Structural Design of Glass for Buildings,” was referenced as the design basis for glass. In the 2015 edition, both CAN/CGSB-12.20-M and ASTM E 1300, “Determining Load Resistance of Glass in Buildings,” are referenced to provide designers with a choice. CAN/CGSB-12.20-M was written in 1989 and has not been updated since then, whereas ASTM E 1300 has been regularly updated since the first edition. Both standards are worth referencing as they each address some applications the other one does not.

Table I-2 compares the requirements of the two standards.

Table I-2
Comparison of CAN/CGSB-12.20-M and ASTM E 1300 Requirements

Design Consideration	Requirements	
	CAN/CGSB-12.20-M	ASTM E 1300
Design basis	Limit states design	Working stress design
Governing code or standard(1)	NBC	ASCE/SEI 7
Load factor(2)	1.5 prior to 2005 NBC 1.4 since 2005 NBC	1.0 until 2010 0.6 since 2010
Reference wind pressure(3)	Mean hourly wind pressure	Based on fastest-mile wind speed until 1998 3 s gust speed since 1998
Specified wind pressures	3 s pressures	3 s pressures
Assumed effective load duration in resistance charts(4)	60 s	60 s prior to 2002 3 s since 2002(5)
Return period of wind loads	10 years in 1989 (1985 NBC) 50 years since 2005 NBC(6)	50 years prior to 2010 in non-hurricane areas 700 years since 2010
Exposure factor	Open terrain only before 2005 NBC Open and suburban terrain since 2005 NBC	Open and suburban terrains
Atmospheric pressure factor for insulating glass units(7)	1.2 factor on wind load	0.95 factor on resistance

(1) ASCE/SEI 7, "Minimum Design Loads for Buildings and Other Structures," is referred to only for the purpose of illustrating changes affecting overall reliability; the reference does not imply that the standard can be used as an alternative to the wind load provisions of the NBC.
(2) In the 2005 edition of the NBC, the load factor was changed from 1.5 to 1.4, which slightly reduced loads. In the 2010 edition of ASCE/SEI 7, the effect of the change in load factor from 1.0 to 0.6 was cancelled out by the change in return period of the reference speed.
(3) Both the NBC and ASCE/SEI 7 have always provided design wind pressures for cladding that were nominally of 3 s duration. In the 1998 edition of ASCE/SEI 7, the fastest-mile reference speeds were changed to 3 s reference speeds, but the pressure coefficients were adjusted so that the calculated design pressures on cladding—which are used with ASTM E 1300—remained the same. Even though the NBC provides reference pressures as mean hourly values, the pressure coefficients for cladding have been calibrated to provide design pressures that are nominally of 3 s duration.
(4) The original justification for using 3 s duration loads with 60 s duration resistance charts (in both CAN/CGSB-12.20-M and ASTM E 1300 loads) was that, during a storm, not only the maximum 3 s load contributes to the degradation of glass resistance: a number of other peak loads of slightly less magnitude also occur in the same storm. The total "effective" duration was originally estimated to be about 60 s. More recent estimates indicate that 60 s is probably conservative and that a more realistic assumption is on the order of 10 s.
(5) The change to 3 s resistance charts in the 2002 edition of ASTM E 1300—while the loads from ASCE/SEI 7 remained unchanged—effectively permitted a reduction from previous levels of reliability.
(6) In the 2005 edition of the NBC, the increase in the return period from 10 to 50 years effectively dictated a higher level of reliability than before, although this was partly offset by the more permissive exposure factor that was introduced at the same time.
(7) The difference in treatment of insulating glass units results in ASTM E 1300 being more permissive by about 15% compared with CAN/CGSB-12.20-M.

In considering the differences in approach of both standards, it was decided that a return to the level of reliability originally achieved in 1989 using the NBC and CAN/CGSB-12.20-M was in order. Over time, the application of CAN/CGSB-12.20-M with the NBC provided more conservative results, while the application of ASTM E 1300 with ASCE/SEI 7 provided less conservative ones. To make the overall level of reliability similar to the one achieved in 1989 using CAN/CGSB-12.20-M, a load adjustment factor of 0.75 on wind load, W, was introduced in Article 4.3.6.1. in the NBC 2015.

This adjustment factor is different from the wind load factor of 1.4, which must nonetheless be applied. Thus, in using CAN/CGSB-12.20-M with the NBC 2015, the value of W to be used with the 60 s resistance charts is calculated as follows:

$$\text{Wind Load Factor} \times \text{ Load Adjustment Factor} \times \mathrm{W} = 1.4 \times 0.75 \times \mathrm{W}$$

To avoid creating a bias between using ASTM E 1300 and CAN/CGSB-12.20-M, it was determined that a load adjustment factor of 1.0 was required on W in using ASTM E 1300 with the NBC 2015. In studies of a number of representative glass configurations, this value produced very similar results to those achieved using CAN/CGSB-12.20-M with a load adjustment factor of 0.75. Thus, in using ASTM E 1300 with the NBC 2015, the value of W to be used with the 3 s resistance charts is calculated as follows:

$$\text{Wind Load Factor} \times \text{ Load Adjustment Factor} \times W = 1.4 \times 1.0 \times W$$

Table I-3 compares the maximum capacities of seven different configurations of double-paned annealed insulating glass units (IGU) calculated using ASTM E 1300 and CAN/CGSB-12.20-M.

Table I-3
Maximum Capacities of Double-Paned Annealed Insulating Glass Units: CAN/CGSB-12.20-M versus ASTM E 1300

IGU Properties						Results According to Standard					
Number of Panes – Thickness of Each Pane, mm	Height, mm	Width, mm	Area, m^2	Aspect Ratio	Thickness/ $Area^{1/2}$	60 s Duration Resistance,(1) kPa	3 s Duration Resistance,(2) kPa	Resistance(3)	Maximum Design Load, kPa	Maximum Design Load, kPa	Maximum Design Load, kPa
						CAN/CGSB	ASTM	CAN/CGSB/ ASTM	CAN/CGSB	ASTM	CAN/CGSB/ ASTM
2 – 6	1000	1000	1.00	1.00	2.45	4.93	6.65	0.74	4.70	4.75	0.99
2 – 6	2000	1000	2.00	2.00	1.22	2.45	3.35	0.73	2.33	2.39	0.98
2 – 6	3000	1000	3.00	3.00	0.82	1.71	2.21	0.77	1.63	1.58	1.03
2 – 6	1500	1500	2.25	1.00	1.09	2.80	3.87	0.72	2.67	2.76	0.96
2 – 6	3000	1500	4.50	2.00	0.54	1.39	1.94	0.72	1.32	1.39	0.96
2 – 6	4500	1500	6.75	3.00	0.36	0.78	1.12	0.70	0.75	0.80	0.93
2 – 8	4500	1500	6.75	3.00	0.42	1.23	1.63	0.75	1.17	1.16	1.01

(1) The 60 s duration CAN/CGSB resistance must be divided by the applicable wind load and load adjustment factors, i.e., 1.4 x 0.75 = 1.05, to obtain the maximum allowable CAN/CGSB design load.

(2) The 3 s duration ASTM resistance must be divided by the applicable wind load and load adjustment factors, i.e., 1.4 x 1.0 = 1.4, to obtain the maximum allowable ASTM design load.

(3) The CAN/CGSB/ASTM resistance value will be 0.75 in any configuration for which the maxima for CAN/CGSB and ASTM are the same.

Although the design specified wind loads and wind load factors are the focus of the design basis for glass, the live load requirements of NBC Section 4.1. also apply to the design of glass panels, including windows, that are installed where the floor elevation on one side of the glass is more than 600 mm higher than the floor or ground on the other side, and to glass that extends below the minimum height of a guard; in such cases, the guard load requirements of NBC Article 4.1.5.14. or 4.1.5.16. are applied in combination with the wind load requirements in accordance with NBC Table 4.1.3.2.-A. Where the height of glass panels, including windows, does not extend below the minimum height of a guard, guard loads need not be considered in the design of the glass.

Vortex Shedding

69. Slender, free-standing cylindrical structures, such as chimneys, observation towers and, in some cases, high-rise buildings, should be designed to resist the dynamic effect of vortex shedding. A structure may be considered slender in this context if the ratio of height to width or diameter exceeds 5. When the wind blows across slender prismatic or cylindrical structures, vortices are shed alternately from one side and then the other along the length of the structure, giving rise to a fluctuating force acting at right angles to the wind direction. The wind speed, V_{Hc}, at the top of the structure when the frequency of vortex shedding equals the natural frequency, f_n, is given by:

$$V_{Hc} = \frac{1}{S} f_n D \qquad (7)$$

where

V_{Hc} = critical mean wind speed at the top of the structure, in m/s, when resonance due to vortex shedding occurs,

S = Strouhal Number, which is dependent on the shape of the cross-section,

f_n = frequency, in Hz, and

D = width or diameter, in m.

For circular and near-circular cylinders, the Strouhal Number is approximately 1/6 for small-diameter structures such as chimneys, and 1/5 for large-diameter structures such as observation towers or buildings. For non-circular cylindrical structures, the Strouhal Number is approximately 1/7.

70. The dynamic effects of vortex shedding of circular and near-circular cylindrical structures, including tapered structures, can be estimated in accordance with Reference [42]. Wind-tunnel tests are recommended for non-circular cylindrical structures (see ASCE/SEI 49).

Lateral Deflection of Tall Buildings

71. Lateral deflection of tall buildings under wind loading may require consideration from the standpoints of serviceability or comfort. The general trend is toward more flexible structures, partly because adequate strength can now be achieved by using higher strength materials that may not provide a corresponding increase in stiffness.

72. One symptom of unserviceability may be the cracking of masonry and interior finishes. Unless precautions are taken to permit movement of interior partitions without damage, a maximum lateral deflection limitation of 1/250 to 1/1 000 of the building height should be observed. According to NBC Sentence 4.1.3.5.(3), 1/500 should be used unless other drift limits are specified in the design standards referenced in NBC Section 4.3. or a detailed analysis is made.

Building Vibration

73. While the maximum lateral wind loading and deflection are generally in the direction parallel to the wind (i.e., the along-wind direction), the maximum acceleration of a building leading to possible human perception of motion or even discomfort may occur in the direction perpendicular to the wind (i.e., the across-wind direction). Across-wind accelerations are likely to exceed along-wind accelerations if the building is slender about both axes, that is, if $\sqrt{wd}/H$ is less than one-third, where w and d are the across-wind effective width and along-wind effective depth, respectively, and H is the height of the building. The along-wind effective depth, d, is calculated using the formula given in NBC Sentence 4.1.7.2.(2) by replacing w_i by d_i.

74. The accelerations in a building are very dependent on the building's shape, orientation and buffeting from surrounding structures. However, data on the peak across-wind acceleration at the top of the building from a variety of turbulent boundary-layer wind-tunnel studies exhibit much scatter around the following empirical formula:

$$a_W = f_{nW}^2 g_p \sqrt{wd}\left(\frac{a_r}{\rho_B g \sqrt{\beta_W}}\right) \tag{8}$$

75. In less slender structures or for lower wind speeds, the maximum acceleration may be in the along-wind direction and may be estimated from the following expression:

$$a_D = 4\pi^2 f_{nD}^2 g_p \sqrt{\frac{KsF}{C_{eH}\beta_D}\frac{\Delta}{C_g}} \tag{9}$$

The variables in the formulae given in Paragraphs 74 and 75 have the following definitions:

w, d = across-wind effective width and along-wind effective depth, respectively, in m,

a_W, a_D = peak acceleration in across-wind and along-wind directions, respectively, in m/s²,

a_r = $78.5 \times 10^{-3}\left[V_H/\left(f_{nW}\sqrt{wd}\right)\right]^{3.3}$, in N/m³,

ρ_B = average density of the building, in kg/m³,

β_W, β_D = fraction of critical damping in across-wind and along-wind directions, respectively,

f_{nW}, f_{nD} = fundamental natural frequencies in across-wind and along-wind directions, respectively, in Hz,
Δ = maximum wind-induced lateral deflection at the top of the building in along-wind direction, in m, at the return period for which accelerations are being evaluated, and
g = acceleration due to gravity = 9.81 m/s^2.

The variables g_p, K, s, F, C_{eH}, and C_g are as defined previously in connection with NBC Sentence 4.1.7.8.(4).

76. Although many additional factors such as visual cues, body position and orientation, and state of mind influence human perception of motion, when the amplitude of acceleration is in the range of 0.5% to 1.5% of g, movement of the building becomes perceptible to most people.[30][43][44]

77. Historically, Equations (8) and (9) have been used with 1-in-10-year wind acceleration limits of 1% to 3% of g for the preliminary assessment of tall buildings. In North America in the period 1975 to 2000, many of the tall buildings that underwent detailed wind tunnel studies were designed for a peak 1-in-10-year acceleration in the range of 1.5% to 2.5% of g. The lower end of this range was generally applied to residential occupancies and the upper end to office occupancies; their performance based on these criteria appears to be generally satisfactory. More recently, other criteria such as the criterion in ISO 10137, "Bases for Design of Structures – Serviceability of Buildings and Walkways against Vibration," have been published that depend on the building's lowest natural frequency and are based on a 1-year return period rather than a 10-year return period. Such criteria, which are expressed as the 1-year peak acceleration, in % of g, evaluated at 10-minute mean wind speed, $V_{10\text{-min},1}$, should not exceed $0.61f_n^{-0.454}$ for office occupancies and $0.41f_n^{-0.454}$ for residential occupancies, where f_n is the lowest natural vibration frequency, in Hz. These formulae are applicable to buildings whose f_n value is less than 1 Hz. For an office occupancy, this results in a 1-year peak acceleration limit of 1.3% of g when f_n = 0.2 Hz and 1.7% of g when f_n = 0.1 Hz. For residential occupancies, the corresponding limits are 0.9% of g and 1.2% of g. The 1-year 10-minute mean wind speed, $V_{10\text{-min},1}$, in m/s, for use with the criterion in ISO 10137, is estimated as follows:

$$V_{10-\min,1} = 1.06\,(V_{10} - 1.45 \times (V_{50} - V_{10})) \qquad (10)$$

where V_{10} and V_{50}, in m/s, are 10-year and 50-year return period values of hourly mean wind speed.

78. Owing to the relative sensitivity of Equations (8) and (9) to the natural frequency of vibration, and of Equation (9) to the corresponding building stiffness, these properties should be determined using fairly rigorous methods, and approximate formulae should be used with caution. For example, the adoption of a natural frequency of 10/N, where N is the number of storeys, may not be consistent with the assumption that the displacement under wind loading is as large as H/500.

Sample Calculation of a_W and a_D

79. A detailed calculation of a_W and a_D using Equations (8) and (9) is shown below. It is based on the sample problem worked out in Paragraph 49 and the following assumptions:

$f_{nW} = f_{nD}$ = 0.2 Hz
q_{10} = 0.37 kPa
$\beta_W = \beta_D$ = 0.015
ρ_B = 176 kg/m^3
$\overline{V}$ (1-in-10-year wind speed) = 23.9 m/s

Step 1: Calculate required parameters.

C_{eH} = 1.9 (from NBC Figure A-4.1.7.8.(2) and (3))
$V_H = \overline{V}\sqrt{C_{eH}}$ (as per NBC Sentence 4.1.7.8.(4))
$= 23.9 \times \sqrt{1.9}$
= 32.9 m/s
w/H = aspect ratio
= 30.5/183
= 0.17
f_n/V_H = wave number for calculation of F (f_n represents f_{nW} or f_{nD} since they are the same and equal to 0.2 Hz)
= 0.2/32.9
= 0.0061

$f_n H/V_H$ = reduced frequency for calculation of s
= 0.2 × 183/32.9
= 0.112

Step 2: Calculate σ/μ (as per NBC Sentence 4.1.7.8.(4)) using the following parameters:
K = 0.10 for Exposure B,
B = 0.62 (from NBC Figure 4.1.7.8.),
s = 0.093 (from NBC Figure A-4.1.7.8.(4)-B),
F = 0.26 (from NBC Figure A-4.1.7.8.(4)-C), and
β = 0.015 (given).

$$\begin{aligned}\sigma/\mu &= \sqrt{\frac{\mathrm{K}}{\mathrm{C_{eH}}}\left(\mathrm{B}+\frac{\mathrm{sF}}{\beta}\right)}\\ &= \sqrt{\frac{0.1}{1.9}\left(0.62+\frac{0.093\times 0.26}{0.015}\right)}\\ &= 0.343\end{aligned}$$

Step 3: Calculate ν (as per NBC Sentence 4.1.7.8.(4)).

$$\begin{aligned}\nu &= \mathrm{f_n}\sqrt{\frac{\mathrm{sF}}{\mathrm{sF}+\beta\mathrm{B}}}\\ &= 0.2\sqrt{\frac{0.093\times 0.26}{0.093\times 0.26+0.015\times 0.62}}\\ &= 0.170/\mathrm{s}\end{aligned}$$

Step 4: Obtain peak factor, g_p.
g_p = 3.74 (from NBC Figure A-4.1.7.8.(4)-A)

Step 5: Calculate C_g (as per NBC Sentence 4.1.7.8.(4)).
C_g = 1 + g_p(σ/μ)
= 1 + 3.74 × 0.343
= 2.28

Step 6: Calculate a_r (as per Paragraph 75).

$$\begin{aligned}\mathrm{a_r} &= 78.5\times 10^{-3}\left[\mathrm{V_H}/\left(\mathrm{f_{nW}}\sqrt{\mathrm{wd}}\right)\right]^{3.3}\\ &= 78.5\times 10^{-3}\left[32.9/\left(0.2\times 30.5\right)\right]^{3.3}\\ &= 20.4\ \mathrm{N/m^3}\end{aligned}$$

Step 7: Calculate a_W (as per Equation (8)).

$$\begin{aligned}\mathrm{a_W} &= 0.2^2\times 3.74\times 30.5\left(\frac{20.4}{176\times 9.81\sqrt{0.015}}\right)\\ &= 0.45\ \mathrm{m/s^2}\end{aligned}$$

Therefore, a_W/g = 0.45/9.81 = 4.6%.

Step 8: Calculate a_D (as per Equation (9)) (value of Δ is usually determined from a structural analysis; in this example, Δ_{10}, the value of Δ for 1-in-10-year wind, is assumed equal to 0.35 m).

$$\begin{aligned} a_D &= 4\pi^2 \times 0.2^2 \times 3.74\sqrt{\frac{0.1 \times 0.093 \times 0.26}{1.9 \times 0.015}}\,\frac{0.35}{2.28} \\ &= 0.264\,\mathrm{m/s^2} \end{aligned}$$

Therefore, a_D/g = 0.264/9.81 = 2.7%.

In this example, the across-wind accelerations clearly overshadow the along-wind accelerations.

Wake Buffeting and Channelling Effects

80. The wind speeds and spectrum of turbulence around a building that is in the wake of nearby upwind buildings can be substantially altered, which in turn alters its response. Upwind buildings can shelter downwind buildings or amplify wind loads. Wake turbulence can alter buffeting loads and may increase the response of dynamically sensitive structures. Wind tunnel tests can be used to study these effects.

Tornadoes

81. Tornadoes account for the greatest incidence of death and serious injury of building occupants due to structural failure and cause considerable economic loss. However, while the probability of tornado occurrence per km^2 can well exceed 1 x 10^{-5} per year, the probability of any one particular building being hit by a tornado is very small (less than 10^{-5} per year[45]). With some exceptions, such as nuclear power plants, it is generally not economical to design buildings for tornadoes beyond what is currently required by NBC Subsection 4.1.7. because of the low risk of loss to individual owners (insurance is cheaper). It is, however, important to provide key construction details for the safety of building occupants. Investigations of tornado-damaged areas in Eastern Canada[46][47] have shown that the buildings in which well over 90% of the occupants were killed or seriously injured by tornadoes did not satisfy the following two key details of building construction:

(a) the anchorage of house floors into the foundation or ground (the floor takes off with the occupants on it), and
(b) the anchorage of roofs down through concrete block walls (the roof takes off and the unsupported block wall collapses onto the occupants).

82. The first detail—the anchorage of house floors—is essentially covered by NBC Article 9.23.6.1. for typical housing with permanent foundations. CSA Z240.10.1, "Site Preparation, Foundation, and Anchorage of Manufactured Homes," contains anchorage recommendations for protecting mobile homes against the effects of tornadoes. The second detail—roof anchorage in block walls—is essentially covered in CSA S304, "Design of Masonry Structures," through limit states requirements for wind uplift and, for the empirical method of masonry design, by Clause F.1.4 of the standard. Deficiency of this construction detail is especially serious for open assembly occupancies because there is nothing inside, such as stored goods, to protect the occupants from wall collapse. For such buildings in tornado-prone areas, it is recommended that the block walls contain vertical reinforcing linking the roof to the foundation.

83. Key details such as those indicated above should be designed on the basis of a factored uplift wind suction of 2 kPa on the roof, a factored lateral wind pressure of 1 kPa on the windward wall, and suction of 2 kPa on the leeward wall.

84. Based on guidance from engineering and meteorological literature on tornadoes and their effects, and a national database of confirmed tornadoes, Figure I-10 defines three thresholds for tornado-prone regions of Canada as follows:

(1) "regions prone to significant tornadoes" are defined as regions where the estimated probability of occurrence of a significant tornado (F2–F5 with 3 s wind gust speeds in excess of 180 km/h) per km^2 per year exceeds 10^{-5};

(2) “regions prone to tornadoes” are defined as regions where the estimated probability of occurrence of a tornado (F0–F2 with 3 s wind gust speeds in excess of 60 km/h) per km^2 per year exceeds 10^{-5}; and

(3) “regions where tornadoes are possible” are defined as regions where tornadoes have been observed, but where the estimated probability of tornado occurrence per km^2 per year is not more than 10^{-5}.

Further guidance can be obtained from Environment and Climate Change Canada at www.climate.weather.gc.ca.

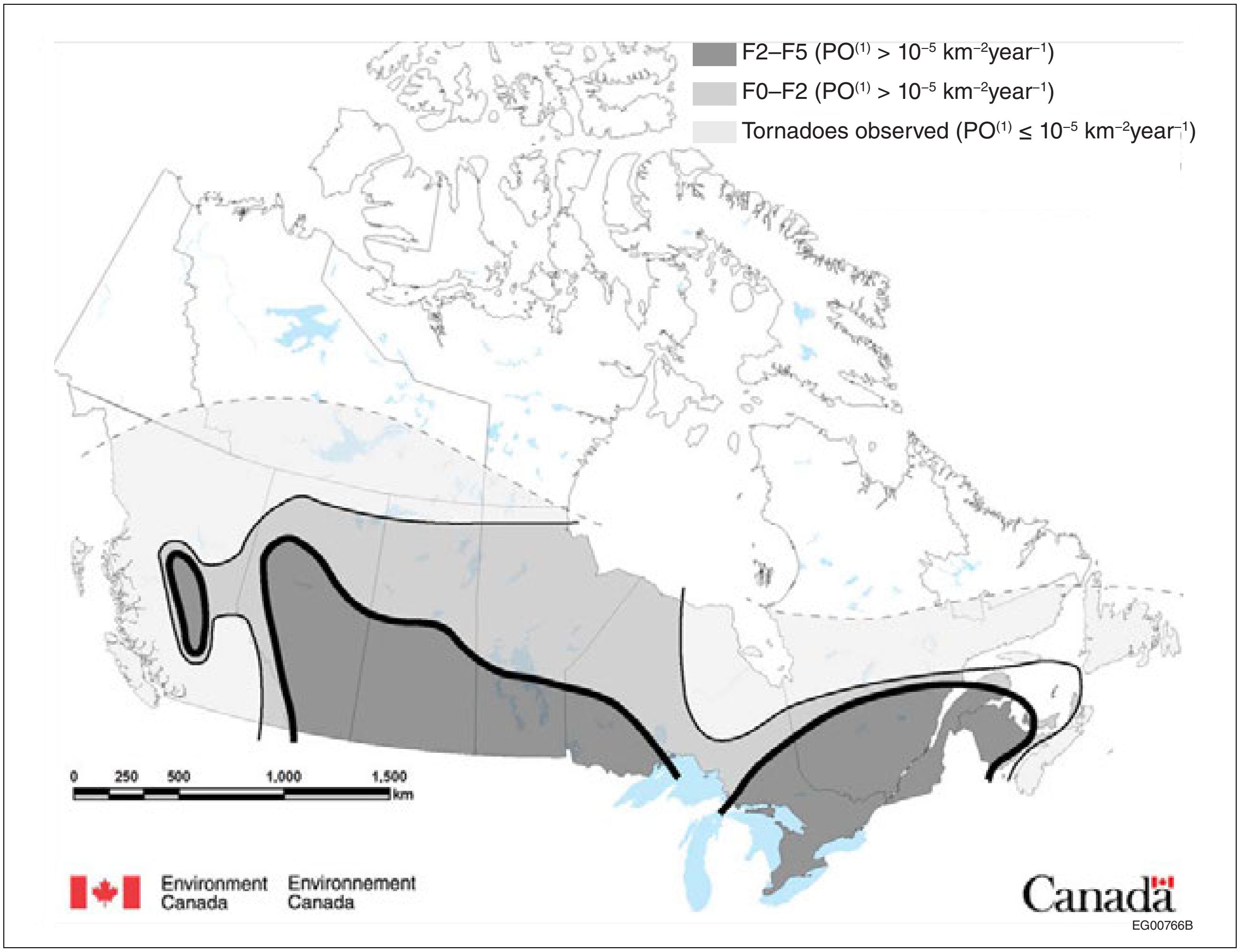


Figure I-10
Tornado-prone regions of Canada using the Fujita (F) scale

Note to Figure I-10:
(1) PO = probability of occurrence

Comparison of Earthquake and Wind Hazards

85. Major changes to the earthquake load provisions in the last three editions of the NBC (2005–2015) included the use of uniform hazard spectrum, the adoption of the 2% probability of exceedance in 50 years (i.e., 2 475-year return period) for specifying the seismic load parameter, and the replacement of the overall force reduction factor by two factors: an explicit overstrength factor, R_o, and a force reduction factor, R_d, which is explicitly linked to the inelastic ductility capacity of the system to sustain cyclic earthquake load.[48][49] Since the seismic design force is reduced by R_dR_o, the design yield force corresponds to a seismic load value with a return period less than 2 475 years. Due to differences in the probabilistic characteristics of the seismic hazard for eastern and western Canada and differences in the ductility capacity of the designed systems, the implied annual probability of incipient yield of the designed structures ranges from about 10^{-2} (1 in 100) to 10^{-3} (1 in 1 000), while the implied annual probability of near incipient collapse of the designed structures is around 4×10^{-4} (1 in 2 500), but with significant variability if the design is governed by a strength requirement.[50][51] These probabilities decrease if a drift requirement governs the structural design or if actual overstrengths are greater than the R_o factor.

The seismic design philosophy on the use of ductile behaviour is currently not used for wind load design.[52][53] The wind load factor of 1.4 in the NBC is calibrated for a tolerable annual failure probability of 3×10^{-5} (1 in 33 000, i.e., a target reliability index of 3 for a 50-year service life) and considers that the nominal wind load is specified based on the 50-year return period value of the hourly mean wind speed.[54][55] The factored design wind load corresponds to a wind load value with a return period of about 500 years, which is resisted by the structure using a factored (reduced) strength. However, with a nominal overstrength of 1.3, a structure can resist a wind load with a return period of about 5 000 to 10 000 years.

The differences in return periods and annual probabilities of occurrence noted above are due to the different shape of the return-period curves. For example, in a very approximate way:
- the wind pressure based on the 50-year-return-period wind is about 60% of the wind pressure based on a 2 500-year-return-period wind;
- the earthquake ground motion based on the 50-year return period is about 15% of the earthquake ground motion based on a 2 500-year return period.

This explains why there seem to be more days of noticeable wind than days of noticeable earthquake ground motions. More detailed discussions are presented by Bartlett et al.[54][55] and DeVall.[56]

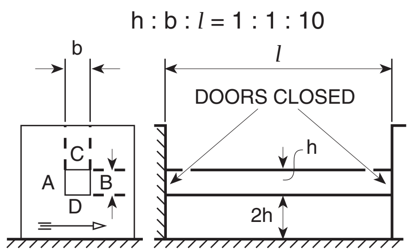
History of Wind Load Provisions in the NBC

86. In 1995, the information on the procedures for obtaining wind pressures, q, for return periods of 10, 30 and 100 years was moved from Chapter 1 of the Supplement to the National Building Code of Canada 1990 to Appendix C of the NBC.

87. In the NBC 2005:
- the three return periods of 10, 30 and 100 years were replaced by one of 50 years;
- an importance factor for wind, I_W, was introduced in the expressions for calculating wind pressures p and p_i;
- under the Static Procedure, a category for rough terrain was added for the calculation of the exposure factor, C_e, and a modified internal gust effect factor, C_{gi} was introduced for the calculation of internal wind pressure;
- a definition for “effective width” was introduced; and
- percentages of wind load removal allowed for partial wind load distribution were increased.

88. In the NBC 2010:
- the building height that triggers the requirement to use the Dynamic or Wind Tunnel Procedure was lowered from 120 m to 60 m;
- a lowest natural frequency of 1 Hz was introduced as a trigger for the use of the Dynamic or Wind Tunnel Procedure;
- a lowest natural frequency of ¼ Hz was introduced as a trigger for the use of the Wind Tunnel Procedure;
- the minimum reference velocity pressure was changed to 0.30 kPa; and
- Exposure C was eliminated.

Figures



C_p : External pressure coefficients

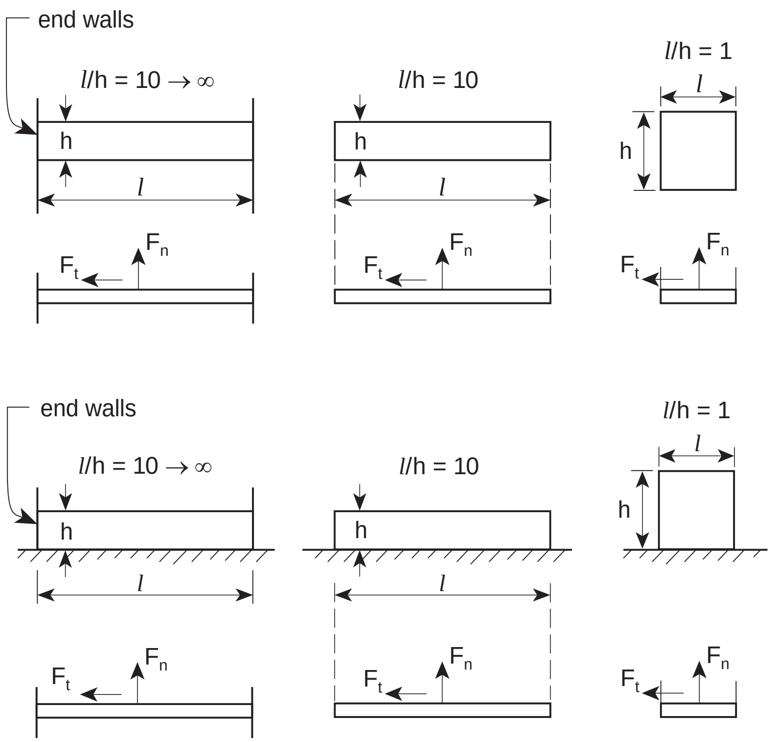
ϕ	A	B	C	D
0°	+0.8	–1.2	–1.4	–1.5

C_{pi} : Internal pressure coefficients

OPENINGS	ϕ = 0°
Uniformly distributed	–0.5
Predominating on side “A”	+0.7
Predominating on side “B”	–1.1
Predominating on side “C”	–1.3

EG00930B

Figure I-11
Closed passage between large walls



C_f, Force Coefficient for Walls Above Ground

l/h	10 → ∞ (End Walls)	10	1
C_f	2.0	1.3	1.15

C_f, Force Coefficient for Walls on the Ground

l/h	10 → ∞ (End Walls)	10	1
C_f	1.3	1.2	1.1

$$F_n = I_w C_f C_n q C_g C_e h l$$
$$F_t = I_w C_f C_t q C_g C_e h l$$

Normal and Transverse Combinations

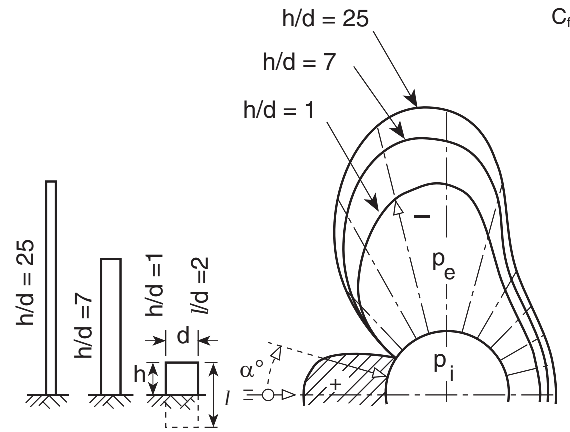
Case	Normal Component, C_n	Transverse Component, C_t
1	1.0	0.2
2	0.6	0.3

EG00931C

Figure I-12
Free-standing plates, walls and billboards

TOTAL FORCE $F = I_w \cdot C_f \cdot q \cdot C_g \cdot C_e \cdot A$, where $A = d \cdot h$

C_f : FORCE COEFFICIENT FOR $d\sqrt{qC_e} > 0.167$



Slenderness h/d= →	25	7	1
Cross section and roughness	C_f	C_f	C_f
Moderately smooth, (metal, timber, concrete)	0.7	0.6	0.5
Rough surface (rounded ribs h = 2%d)	0.9	0.8	0.7
Very rough surface (sharp ribs h = 8%d)	1.2	1.0	0.8
Smooth and rough surface sharp edges	1.4	1.2	1.0

C_p : EXTERNAL PRESS. COEFF. FOR $d\sqrt{qC_e} > 0.167$ and moderately smooth surface

h/d	*l*/d	α=	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
25	50	C_p	+1.0	+0.8	+0.1	–0.9	–1.9	–2.5	–2.6	–1.9	–0.9	–0.7	–0.6	–0.6	–0.6
7	14	C_p	+1.0	+0.8	+0.1	–0.8	–1.7	–1.6	–2.2	–1.7	–0.8	–0.6	–0.5	–0.5	–0.5
1	2	C_p	+1.0	+0.8	+0.1	–0.7	–1.2	–1.6	–1.7	–1.2	–0.7	–0.5	–0.4	–0.4	–0.4

$\Delta p = p_i - p_e$

$p_i = C_{pi} \cdot q \cdot C_g \cdot C_e$

$p_e = C_p \cdot q \cdot C_g \cdot C_e$

Stack fully operating C_{pi} = +0.1; Stack throttled Cpi = –0.8

EG00932C

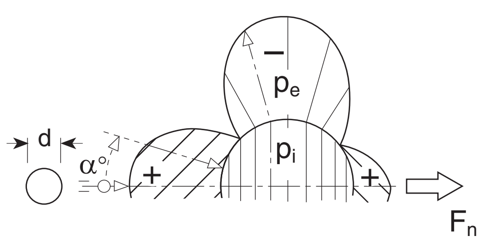
Figure I-13
Cylinders, chimneys and tanks

TOTAL FORCE $F = I_w \cdot C_f \cdot q \cdot C_g \cdot C_e \cdot A$; $A = \dfrac{\pi d^2}{4}$

for $d\sqrt{qC_e} > 0.8$ and moderately smooth surface

C_f : FORCE COEFFICIENT

$C_f = 0.2$



$p = p_i - p_e$

p_i for closed tanks = working pressure

$p_e = C_p \cdot q \cdot C_g \cdot C_e$

Cp : EXTERNAL PRESS. COEFF. FOR $d\sqrt{qC_e} > 0.8$ and moderately smooth surface

α=	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
C_p	+1.0	+0.9	+0.5	–0.1	–0.7	–1.1	–1.2	–1.0	–0.6	–0.2	+0.1	+0.3	+0.4

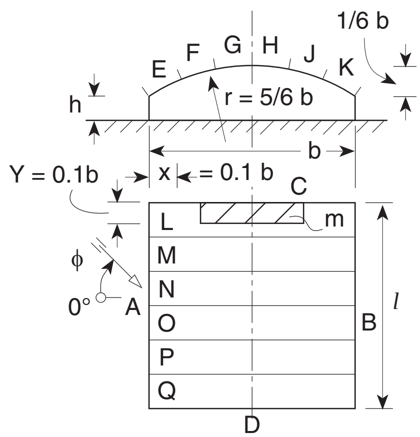
EG00933C

Figure I-14
Spheres

Note to Figure I-14:

(1) The full range of possible internal pressures should be considered for p_i.

RAD. r = 5/6 b   h:b:l = 1:12:12



Hatched Area to Scale

C_p : EXTERNAL PRESSURE COEFFICIENTS

ϕ	A	B	C	D	E	F	G	H	J	K
0°	+0.7	–0.2	–0.3	–0.3	–0.1	–0.5	–0.8	–0.8	–0.4	–0.1
30°	+0.6	–0.3	+0.2	–0.4	–0.1	–0.4	–0.7	–0.9	–0.7	–0.4

ϕ	A	B	C	D	L	M	N	O	P	Q
90°	–0.3	–0.3	+0.9	–0.3	–0.8	–0.7	–0.5	–0.3	–0.1	–0.1
30°	Section “m” $C_p^* = –1.8$ with C_p^* min. = –2.5									

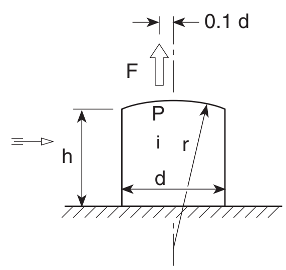
C_{pi} : INTERNAL PRESSURE COEFFICIENTS

OPENINGS	ϕ = 0°	ϕ = 30°	ϕ = 90°
Uniformly distributed	±0.2	±0.2	±0.2
Window Y open on side “A”	+0.4	+0.7	–1.0
All doors open on side “C”	–0.1	+0.6	+0.8
Only door X open on side “C”	–1.5	+0.7	+0.4

EG00934B

Figure I-15
Hangar, curved roof with moderately smooth surface

h : d : r = 1 : 1 : 1.5



Total force on roof $F = (p_i - p_e)\, A I_w$

p_i = working pressure in kN/m²

$p_e = C_p \cdot q \cdot C_g \cdot C_e$

$A = \frac{\pi}{4} d^2$

External pressure coefficient $C_p = –1.0$

EG00935C

Figure I-16
Roof load on smooth closed tank

C_f = FORCE COEFFICIENTS

$l/d > 100$

Total force $F = C_f \cdot q \cdot C_g \cdot C_e \cdot A \cdot I_w$

$A = d \cdot l$

	$d\sqrt{q\,C_e}$ < 0.167	$d\sqrt{q\,C_e}$ > 0.167
Smooth wires, rods, pipes	1.2	0.5
Mod. smooth wires and rods	1.2	0.7
Fine wire cables	1.2	0.9
Thick wire cables	1.3	1.1

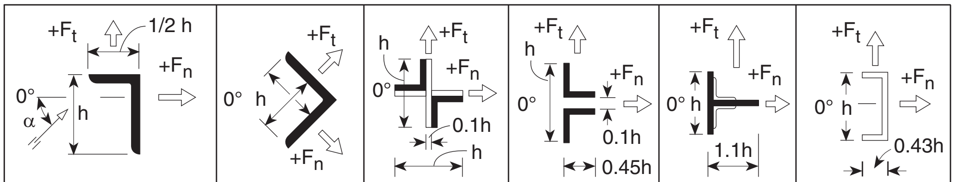
EG00936C

Figure I-17
Poles, rods and wires

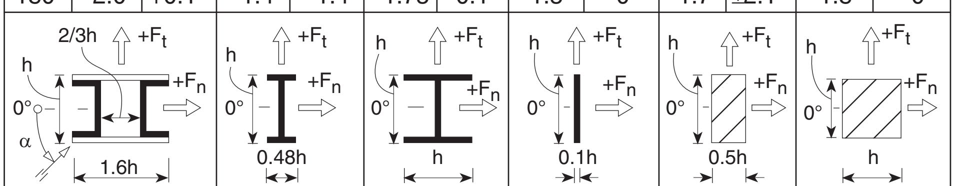
l = Length of member
$A = h \cdot l$ = Area

For wind normal to axis of member: Normal force $F_n = k \cdot C_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A \cdot I_w$
Tangential force $F_t = k \cdot C_{t\infty} \cdot q \cdot C_g \cdot C_e \cdot A \cdot I_w$

$C_{n\infty}$ and $C_{t\infty}$: Force coefficients for an infinitely long member

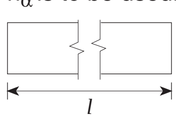


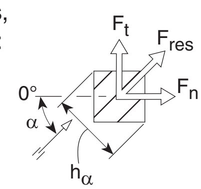
α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1.9	+0.95	+1.8	+1.8	+1.75	+0.1	+1.6	0	+2.0	0	+2.05	0
45°	+1.8	+0.8	+2.1	+1.8	+0.85	+0.85	+1.5	–0.1	+1.2	+0.9	+1.85	+0.6
90°	+2.0	+1.7	–1.9	–1.0	–0.1	+1.75	–0.95	+0.7	–1.6	+2.15	0	+0.6
135°	–1.8	–0.1	–2.0	+0.3	–0.75	+0.75	–0.5	+1.05	–1.1	+2.4	–1.6	+0.4
180°	–2.0	+0.1	–1.4	–1.4	–1.75	–0.1	–1.5	0	–1.7	±2.1	–1.8	0



α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1.4	0	+2.05	0	+1.6	0	+2.0	0	+2.1	0	+2.0	0
45°	+1.2	+1.6	+1.95	+0.6	+1.5	+1.5	+1.8	+0.1	+1.4	+0.7	+1.55	+1.55
90°	0	+2.2	±0.5	+0.9	0	+1.9	0	+0.1	0	+0.75	0	+2.0

For slenderness, h_α is to be used:





k : Reduction factor for members of finite slenderness (in general use full length not panel length)

l/h_α	5	10	20	35	50	100	∞
k	0.60	0.65	0.75	0.85	0.90	0.95	1.0

EG00937C

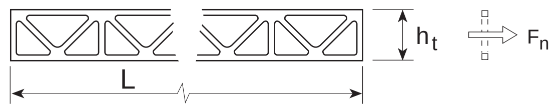
Figure I-18
Structural members, single and assembled sections

A_s = Solid area of the truss

$A = h_t \cdot L$

A_s / A = Solidity ratio

For wind normal to surface A : Normal force $F_n = k \cdot C_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s \cdot I_w$



$C_{n\infty}$: Force coeff. for an infinitely long truss, $0 \leq A_s/A \leq 1$

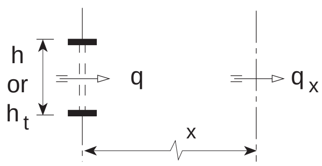
A_s/A	0	0.1	0.15	0.2	0.3 to 0.8	0.95	1.0
$C_{n\infty}$	2.0	1.9	1.8	1.7	1.6	1.8	2.0

k : Reduction factor for trusses of finite length and slenderness

L/h_t \ A_s/A	0.25	0.5	0.9	0.95	1.0
5	0.96	0.91	0.87	0.77	0.60
20	0.98	0.97	0.94	0.89	0.75
50	0.99	0.98	0.97	0.95	0.90
∞	1.0	1.0	1.0	1.0	1.0

EG00938C

Figure I-19
Plane trusses made from sharp-edged sections



$q_x = k_x \cdot q$

k_x SHIELDING FACTOR

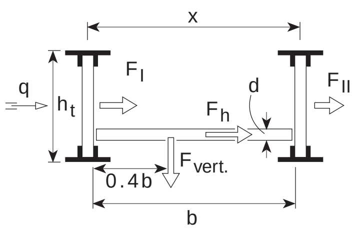
x/h \ A_s/A	0.1	0.2	0.3	0.4	0.5	0.6	0.8	1.0
0.5	0.93	0.75	0.56	0.38	0.19	0	0	0
1	0.99	0.81	0.65	0.48	0.32	0.15	0.15	0.15
2	1.00	0.87	0.73	0.59	0.44	0.30	0.30	0.30
4	1.00	0.90	0.78	0.65	0.52	0.40	0.40	0.40
6	1.00	0.93	0.83	0.72	0.61	0.50	0.50	0.50

EG00939B

Figure I-20
Shielding factors

Note to Figure I-20:
(1) A_s/A = solidity ratio, where A_s is the solid area of the truss and A is the total area of the truss (see also Figure I-19).

CASE I WITHOUT VEHICLES



L_B = Length of bridge

k, $C_{n\infty}$, A_s, k_x (1)

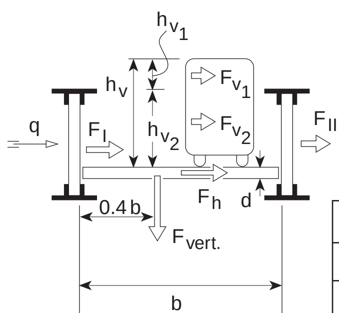
Windward girder $F_I = kC_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s \cdot I_W$

Leeward girder $F_{II} = kC_{n\infty} \cdot k_x q \cdot C_g \cdot C_e \cdot A_s \cdot I_W$

Deck horiz. load $F_h = 1.0 \cdot q \cdot C_g \cdot C_e \cdot L_B \cdot I_W$

Deck vert. load $F_{vert.} = 0.6 \cdot q \cdot C_g \cdot C_e \cdot b \cdot L_B \cdot I_W$

CASE II WITH VEHICLES



L_v = Length of vehicle; $A_1 = h_{v_1} \cdot L_v$

$A_2 = h_{v_2} \cdot L_v$

Windward girder $F_I = kC_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s \cdot I_W$

Leeward girder $F_{II} = kC_{n\infty} \cdot k_x q \cdot C_g \cdot C_e \cdot A_s \cdot I_W$

Deck horiz. load $F_h = 1.2 \cdot q \cdot C_g \cdot C_e \cdot d \cdot L_B \cdot I_W$

Deck vert. load $F_{vert.} = 0.8 \cdot q \cdot C_g \cdot C_e \cdot b \cdot L_B \cdot I_W$

Traffic load $F_{v_1} = C_n q \cdot C_g \cdot C_e \cdot A_1 \cdot I_W$

$F_{v_2} = C_n \cdot 2/3\, q \cdot C_g \cdot C_e \cdot A_2 \cdot I_W$

Height and force coefficients	h_v	C_n
Railway vehicle	3.8 m	1.5
Highway vehicle	3.0 m	1.2
Pedestrian	1.7 m	1.0

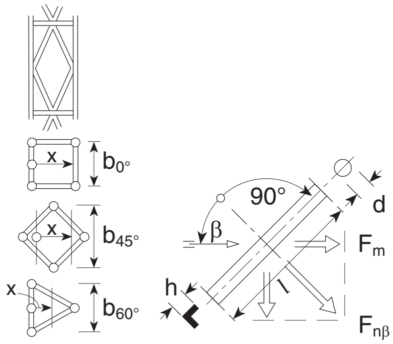
EG00940C

Figure I-21
Truss and plate girder bridges

Note to Figure I-21:

(1) The values for these coefficients are taken from Figures I-18 and I-19.

$A_s/A \leq 0.3$



$A = d \cdot L$ or $h \cdot L$

L = true length of member

β = angle formed by wind direction and the normal to member axis

k_x = a function of A_s/A and x/b

TOTAL LOAD IN WIND DIRECTION $F = \Sigma F_m$

F_m = FORCE ON MEMBER

$F_m = k \cdot C_{\infty\beta} \cdot q \cdot C_g \cdot C_e \cdot A \cos\beta \cdot I_w$

Shielded member

$F_m = k \cdot C_{\infty\beta} \cdot k_x q \cdot C_g \cdot C_e \cdot A \cos\beta \cdot I_w$

Coeff. $C_{\infty\beta}$: For sharp-edged members $C_{\infty\beta} = k_\beta \cdot C_{n\infty}$ and $k_\beta \cdot C_{t\infty}$

Coeff. $C_{\infty\beta}$, k_β, k, k_x(1)

β	SHARP-EDGED MEMBERS			ROUND MEMBERS, SMOOTH and ROUGH SURFACES, $d\sqrt{qC_e} < 0.167$			ROUND MEMBERS, MODERATELY SMOOTH SURFACES, $d\sqrt{qC_e} < 0.167$		
	k_β	k	k_x	$C_{\infty\beta}$	k	k_x	$C_{\infty\beta}$	k	k_x
0°	1.00			1.20			0.60		
15°	0.98			1.16			0.58		
30°	0.93	(2)	(3)	1.04	(2)	(3)	0.53	0.9 for l/d = 25	0.95 constant
45°	0.88			0.85			0.42		
60°	0.80			0.60			0.28		

EG00941C

Figure I-22
Three-dimensional trusses

Notes to Figure I-22:
(1) See Figure I-18 for $C_{n\infty}$ and $C_{t\infty}$ values.
(2) See Figure I-18.
(3) See Figure I-20.

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Commentary J

Design for Seismic Effects

Commentary J

Design for Seismic Effects

Scope

1. The requirements of Subsection 4.1.8. of the National Building Code of Canada (NBC) 2015 apply only to the seismic design of new buildings and should not be used for special structures such as bridges, towers, dams and storage tanks (for recommendations on free-standing storage tanks, see Paragraph 247). However, the effects of tanks within buildings are addressed in NBC Article 4.1.8.18. NBC Subsection 4.1.8. is not specifically intended for the evaluation and upgrading of existing buildings, but the concepts and methods of analysis and design presented therein are often applicable to that purpose as well, as discussed in Commentary L.

2. Even though design forces for wind may be greater than seismic design forces in some situations (i.e., wind "governs" the design), seismic detailing may be required. Even if wind forces govern, the design must accommodate at least the requirements for the type of lateral-load-resisting system and the detailing that correspond to the seismic forces calculated for the building.

Seismic Design Objectives and Expected Performance

3. Earthquakes can cause damage to buildings through any of the following: ground shaking, soil failures caused by shaking (including lateral spreading and settlement caused by liquefaction and slope instability), effects of surface fault ruptures on structures, or tsunamis. The only one of these hazards that is directly addressed by the NBC is ground shaking, although the potential for liquefaction and slope instability and their consequences with respect to buildings are taken into account in the design of the structure and its foundations; the other hazards of surface fault ruptures, landslides and tsunamis are addressed primarily through planning and site selection. Seismic design has the following intents, which are consistent with the overall objectives of the NBC:
(1) to protect the life and safety of building occupants and the general public as the building responds to strong ground shaking,
(2) to limit building damage during low to moderate levels of ground shaking, and
(3) to increase the likelihood that post-disaster buildings can continue to be occupied and functional following strong ground shaking, though low levels of damage can be expected in such buildings.

4. According to the NBC, strong ground shaking is considered to be a rare occurrence in Canada; indeed, NBC Article 4.1.2.1. defines earthquake loads as rare loads. In the Commentary section titled Seismic Hazard (starting at Paragraph 38), strong ground shaking is defined in terms of the mean ground motion amplitude having a probability of exceedance of 2% in 50 years (or approximately 0.1% in 2 years or 4% in 100 years), which corresponds to an annual rate of exceedance of 1/2 475. The ground motion at the 2%-in-50-year level may be termed the maximum earthquake ground motion to be considered, or more simply, the design ground motion (DGM). However, stronger ground shaking can occur, which can be accommodated by engineering beyond the Code requirements.

5. The primary objective of seismic design is to provide an acceptable level of safety for building occupants and the general public as the building responds to strong ground motion; in other words, to minimize loss of life. This implies that, although there may be extensive structural and non-structural damage during the DGM, there is a reasonable degree of confidence that the building will not collapse nor will its attachments break off and fall on people near the building. This performance level is termed "extensive damage" because, although the structure may have lost a substantial amount of its initial strength and stiffness, it retains a margin of resistance against collapse. However, some buildings may be damaged to such an extent that they have to be demolished.

6. A high degree of life safety protection that is consistent with the low probability of the "extensive damage" performance level is achieved through inelastic energy dissipation, which is the explicit seismic design approach used in the NBC seismic provisions. Inelastic energy dissipation leads to a reduction of design forces. Seismic Force Resisting Systems (SFRSs) that do not have a significant inelastic energy dissipation capacity, i.e., those with limited ductility, are subject to higher loads and have less stringent detailing requirements; in certain cases, systems with limited ductility may not be permitted in regions where the DGM is high. The capacity of various kinds of SFRS to resist the anticipated seismic loads is achieved by applying the design and detailing provisions contained in the NBC and in the referenced material standards. There is an enhanced confidence that the integrity of the overall structure will be maintained following strong ground shaking when limits on interstorey drift are imposed that are consistent with the "extensive damage" performance objective.

7. While the foregoing text describes the primary objective of seismic design, designers often wish to have more information on the performance expectations associated with that objective, particularly at levels of ground shaking that approach or exceed the DGM. Heidebrecht[1] discusses such objectives and describes a number of building design features that contribute significantly toward meeting the intended performance objectives. In addition to energy dissipation characteristics, the important features include: regularity of building configuration, overstrength, and reserve ductility capacity in structural elements and joints. Experience during past earthquakes has shown that ductile structures with regular configurations in which the energy dissipation is distributed throughout the structure can sustain their integrity at ground motions considerably higher than the DGM level (Hall[2] and Park et al.[3]). On the other hand, irregular structures with limited ductility often perform poorly at the DGM level because the energy dissipation and damage is concentrated in one part of the structure (e.g., soft-storey structures). Heidebrecht[1] also discusses the important role of capacity design in improving performance expectations at unexpectedly high levels of earthquake ground motion.

8. Design that complies with the NBC provisions is also expected to limit damage at ground shaking levels that are well below the DGM level. When the peak ground motion amplitude is less than half of the DGM level, well-designed and -detailed structures can be expected to sustain limited structural damage (Heidebrecht[1]). Because the primary design objective is based on inelastic energy dissipation, it is implicit that some structural damage can be expected when peak ground motions approach the DGM level. Damage can be reduced by selecting a structural system that has sufficient stiffness to ensure that drifts are below the specified drift limits, which are actually intended to limit the probability of collapse and to limit catastrophic damage. Damage to non-structural elements can be minimized by limiting their deflections, limiting the interstorey drift in the structure, paying careful attention to detailing, providing adequate clearances from the structure, and protecting elements tied rigidly to the structure from deformations, which could cause cracking.

9. The performance objectives for post-disaster buildings differ from those stated above because such buildings must remain operational immediately following an earthquake. Therefore, the performance objective for post-disaster buildings when subjected to DGM-level shaking can best be described as "immediate occupancy." Any damage to the structural system should not impede the continued use and occupancy of the building, and any damage to non-structural systems should be minor; the structure is expected to retain most of its pre-earthquake strength and stiffness; mechanical, electrical, plumbing and other systems necessary for normal operation are expected to remain functional. This more stringent performance objective is achieved in two ways:
   (1) through the use of an importance factor (1.5 for post-disaster buildings and 1.3 for buildings in the High Importance Category, e.g., schools and community centres, that are likely to be used as post-disaster shelters) to increase the design lateral load, and
   (2) through the establishment of a much lower interstorey drift limit.

   Other factors, such as building configuration, type of structural framing, materials and as-built construction details, have a significant effect on the ability of the building to achieve this performance objective. The NBC incorporates some of these considerations by prohibiting most structural irregularities in locations having moderate to high levels of DGM and by requiring that the SFRS have a minimum ductility capacity.

10. When considering performance objectives, it is important to recognize that the wide range of possible building characteristics, site conditions and earthquake characteristics will contribute to a very wide range of actual performance during any future earthquake. Although the NBC seismic provisions are intended to provide an acceptable level of protection, experience and observations during past earthquakes elsewhere in the world have shown that a wide variation in the extent of damage can be expected during any future earthquake event. Irregular structures and those with poor detailing can be expected to perform poorly, while regular, well-designed and -detailed structures can be expected to perform considerably better. Although the minimum requirements in these provisions are intended to provide an adequate level of protection against collapse, good performance is best achieved when designers follow coherent design and detailing approaches that are consistent with the intent of the NBC seismic provisions and the building is constructed according to that design.

Rationale for Updating the NBC 2010 Seismic Provisions

11. One of the major reasons for revising the NBC 2010 seismic provisions was to incorporate the ongoing improvement in knowledge on seismic hazard and its geographical distribution throughout the country. The NBC 2010 provisions were based on data and knowledge from the 1990s. Since then, a large amount of new data has been collected, and new Ground Motion Prediction Equations (GMPEs) have been developed. A history of how seismic hazard information was used for the determination of seismic design forces in the 2010 and earlier editions of the NBC can be found in Table J-1 of Commentary J in the "User's Guide – NBC 2010, Structural Commentaries (Part 4 of Division B)."

12. There are several other major reasons for updating the NBC seismic provisions over and above those directly related to seismic hazard. First, studying and learning from the damage caused by major earthquakes around the world allows engineers to determine whether or not current Code provisions can provide an adequate level of protection in buildings and other facilities being constructed in Canada. Each major earthquake provides one or more significant lessons that may lead to further Code improvements.

13. Another reason for the periodic updating of the NBC seismic provisions is to take into account the results of broadly based earthquake engineering research being conducted in Canada and around the world.

14. A further reason for updating the NBC seismic provisions is to be responsive to the changes made in foreign codes. Canadians benefit from the experience and research used to make changes in other codes; when analysis of such developments shows that the NBC provisions could be improved, then the developments are adapted for use in the NBC.

Summary of Major Changes in the NBC 2015

- Complete revision and updating of seismic hazard information on the basis of a large amount of new data and new GMPEs
- Modification of the approach to site coefficients for ground motion amplification
- Modification of the short-period cut-off for the determination of the minimum lateral earthquake design force
- Introduction of a simplified analysis procedure for regions of low seismicity that also serves as a minimum level of design for structural integrity
- Introduction of a new type of irregularity, which addresses gravity-induced lateral demand
- Several additions and modifications to NBC Table 4.1.8.9., including the addition of entries for tilt-up structures and a table note on industrial steel structures
- Introduction of requirements relating to irregularities and the design force level for buildings constructed with more than 4 storeys of continuous wood construction
- Introduction of requirements relating to the fundamental lateral period and the effects of in-plane diaphragm deformations for single-storey buildings with flexible roof diaphragms of steel deck or wood
- Revision of higher mode factors and base overturning reduction factors
- Modification of the foundation provisions, including those that address foundation movements and their effect on the structure
- Introduction of requirements for the design of glass, elevators and pallet racks

- Introduction of requirements for the design of seismically isolated structures and structures with supplemental energy dissipation systems

Seismic Hazard in Previous Editions of the NBC

15. Seismic hazard in the 1985, 1990 and 1995 editions of the NBC was described in terms of peak ground velocity, v, and peak ground acceleration, a, determined at a probability of exceedance of 10% in 50 years. The period-dependent variation of seismic forces was obtained by multiplying v by a seismic response factor, denoted S in the NBC 1995, with the shape of S dependent upon the ratio of a to v. Uniform hazard spectral values—i.e., spectral response acceleration ordinates at different periods calculated at the same probability of exceedance—provide a much better period-dependent representation of earthquake effects on structures and have been used in the NBC since 2005. The Geological Survey of Canada provides selected spectral response acceleration values for specific geographical locations in Canada. Because the spectral response acceleration ordinates are determined directly at each geographical location, the differences in spectral shape across the country are reflected directly in the determination of design forces, rather than being approximated by using zonal values of peak ground velocity and acceleration.

For the 1985, 1990 and 1995 editions of the NBC, the probability level used to define the DGM was the 10%-in-50-year probability of exceedance, which corresponds to a return period of 475 years. For the NBC 2005, the probability of exceedance was reduced to 2% in 50 years (corresponding to a return period of 2 475 years), a probability level that is consistent with the "rare events" for which extensive damage, short of collapse, is tolerable (see the section titled Change in Return Period (Probability of Exceedance) in Commentary J of the "User's Guide – NBC 2010, Structural Commentaries (Part 4 of Division B)" for additional details). This probability level is retained for the NBC 2015.

Mean hazard results are used in the NBC 2015 instead of the median hazard results used in the NBC 2010. The mean hazard values are generally higher than the corresponding median hazard values. Using mean hazard values results in about 1 chance in 3 that the actual ground motion for a specified probability of exceedance will exceed the DGM value and a lower likelihood that the actual ground motion will greatly exceed the DGM value.

Seismic hazard values for selected higher probabilities can be found by specifying the latitude and longitude of a particular location in the "Hazard Calculator" on the Earthquakes Canada Web site of the Geological Survey of Canada (www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php). The probability of exceedance in n years, P(n), corresponding to an annual rate of exceedance, λ, (which is equal to the inverse of the return period in years) can be expressed as $P(n) = 1-e^{-\lambda n}$. Alternatively, the approximation $P(n) = 1-(1-\lambda)^n$, which is suitable for the return periods relevant for building code purposes, can be used.

Major Changes to the NBC 2015 Seismic Hazard Model

Use of a Fully Probabilistic Model

16. The model used to generate the seismic hazard values for the NBC 2015 is fully probabilistic. It replaces the "robust" quasi-probabilistic model used for the NBC 2005 and NBC 2010, which chose the largest of the median hazard amplitudes resulting from three alternative probabilistic source zone models and from a deterministic model for the Cascadia subduction earthquake scenario as the DGM (Adams et al.[4]). Adoption of the fully probabilistic model was made possible by improved understanding of the seismotectonics and improved quantification of the uncertainty.

Use of Mean Hazard Ground Motion Amplitudes

17. In the context of the fully probabilistic model, the use of mean hazard ground motion amplitudes was adopted for the NBC 2015, as explained further in Paragraph 58. The mean hazard results used in the NBC 2015 better reflect the expected seismic hazard than did the median hazard results used in the NBC 2010.

Inclusion of Active Faults

18. The NBC 2005 seismic hazard model included a probabilistic treatment of one fault source (the Queen Charlotte fault) and a deterministic treatment of the Cascadia subduction zone. For the NBC 2015, the probabilistic model includes fault sources for three low-angle subduction thrusts in the Cascadia subduction zone, an updated treatment of the fault sources offshore of Haida Gwaii (formerly the Queen Charlotte Islands) and five onshore fault sources of strike-slip or reverse type in the Yukon–Alaska region. The inclusion of the five onshore fault sources appropriately concentrates the earthquake occurrence near the faults, instead of averaging it out over a wider area (as was done for the NBC 2010). The seismic hazard predicted by the 2015 model is much higher close to these faults than away from them (i.e., there is a steep gradient in hazard away from the fault). For sites very close to the modeled onshore faults, the predicted seismic hazard is now much higher than in the NBC 2010. Design guidance for sites in the Yukon close to active faults is given in Paragraph 94.

Period-Dependent Site Coefficients

19. It has long been recognized that the amplification of seismic motions from rock to soil sites can be significant, especially at sites with soft soil conditions. Borcherdt[5] and others developed a procedure to quantify the effects of soil conditions on the seismic response of a site, which was adopted for the NBC 2005 and NBC 2010, as discussed by Finn and Wightman.[6] The procedure involved the categorization of soil profiles according to quantitative measures of soil properties (shear wave velocity, standard penetration resistance or undrained shear strength), the effects of the intensity of underlying rock motion, and the amplification of surface motions. The site effects were represented by two site coefficients, one for short-period average spectral response and the other for mid-period average spectral response.

20. In recent years, many more instrumental records of the seismic response of a wide variety of soil sites have been gathered. This expanded database has allowed seismologists to separate the amplification or de-amplification of soil sites of various consistencies into period bands and incorporate this in GMPEs. Thus, site coefficients for spectral response acceleration at a number of periods, for Peak Ground Acceleration (PGA), and for Peak Ground Velocity (PGV) are now available, rather than site coefficients for short- and mid-period average spectral response only. The period-dependent site coefficients adopted for the NBC 2015 stem from the original work of Choi and Stewart,[7] simplified somewhat and adapted for use by Boore and Atkinson[8] in their 2008 GMPE. The equations for the site coefficients contain a linear term and a non-linear term. These terms are functions of the PGA predicted for the B/C Site Class boundary, of the average shear wave velocity, $\bar{V}_{s30}$, for each Site Class, and of a set of period-dependent regression coefficients. The site coefficients given in NBC Tables 4.1.8.4.-B to -I were determined using the Boore and Atkinson[8] site amplification equations to represent the mean amplification or de-amplification for the $\bar{V}_{s30}$ ranges given in NBC Table 4.1.8.4.-A.

Delineation of Effects of Overstrength and Ductility

21. The NBC seismic provisions have long taken into account, either implicitly or explicitly, that seismic forces are reduced when structural response goes into the inelastic range. This is an important property that enables structures to resist strong earthquake shaking, provided they have the capacity to deform inelastically through several load reversals without a significant loss of strength. In the NBC 2005, a ductility-related force modification factor, R_d, was introduced in the denominator of the expression used to calculate the lateral earthquake design force, V. In the NBC 2015, values of R_d are given for a wider range of structural systems than in the NBC 2010.

22. It has been well recognized that various features of structural systems and their design (e.g., material factors used in design, minimum design requirements, capacity design, load combinations and the redistribution of forces arising from redundancy) often lead to a lateral strength that is considerably larger than that used as the basis for design. As such, in the NBC 2005, an explicit overstrength-related force modification factor, R_o, was introduced in the denominator of the expression used to calculate the lateral earthquake design force, V. This factor is intended to represent the minimum level of overstrength that can be counted on for each particular SFRS. In the NBC 2015, the value of R_o ranges from 1.0 to 1.7.

23. The rationale for the use of the factors R_d and R_o and an explanation of the particular values or ranges of values for various structural systems are given by Mitchell et al.[9]

Period Calculations

24. The calculation of the fundamental lateral period, T_a, is significant because its value determines the spectral response acceleration at that period, $S(T_a)$. On the one hand, the determination of T_a needs to be relatively simple; on the other hand, its value should not be overestimated. Values of T_a that are larger than a realistic value can be expected to result in an underestimate of the lateral earthquake design force, V, and an overestimate of lateral deflection.

25. While the empirical formulae for calculating the periods of moment-resisting frames, steel-braced frames, shear walls and other structures have remained unchanged in the NBC 2015, new empirical formulae have been introduced for calculating the periods of single-storey buildings with flexible roof diaphragms of steel deck or wood. It is important to note that the period determined from the empirical formulae is to be used only for the calculation of V and not in the calculation of wind forces.

26. The NBC provisions allow period calculations using other established methods of mechanics, but an upper limit is applied to the periods so calculated instead of placing a lower limit on the lateral earthquake design force, V. For moment-resisting frames and single-storey buildings with flexible roof diaphragms of steel deck or wood, the upper limit is 1.5 times the period determined using the applicable empirical formula, and for braced frames and shear walls, the upper limit is 2.0 times the period determined using the applicable empirical formula. The imposition of an upper limit on the fundamental lateral periods of structures is justified because of the concern that structural models frequently overestimate the flexibility of a structural system (e.g., by neglecting non-structural stiffening elements), giving rise to an overestimate of the natural period.

Higher Mode Effects in the Equivalent Static Force Procedure

27. The Equivalent Static Force Procedure (ESFP) used to calculate the lateral earthquake design force, V, in the NBC provisions and in other codes is based on the assumption that the main features of the dynamic response of the structure can be represented by a single mode response at the fundamental lateral period, T_a. However, many structures, particularly those with longer periods, have significant higher mode effects, which are taken into account by modifications to both the value of V and the distribution of the shears and moments along the height of the structure.

28. Since the NBC 2005, higher mode effects have been represented by an additional force applied at the top of the structure, F_t, an overturning moment reduction factor, J, and a higher mode factor, M_v. In the NBC 2015, the values of M_v and J have been revised in accordance with the changes to the seismic hazard values. These factors are now listed in NBC Table 4.1.8.11. for four values of the spectral ratio, i.e., the ratio of the design spectral response acceleration values at periods of 0.2 s and 5.0 s, S(0.2)/S(0.5), and are interpolated for intermediate values of the spectral ratio. A more detailed discussion is provided in the Commentary section on NBC Sentence 4.1.8.11.(6) (starting at Paragraph 163).

29. The simulation of higher mode effects in the ESFP is not valid for structures with long periods because their response may be dominated by the second or even third mode; the ESFP only takes account of higher mode effects when the fundamental mode dominates response. Consequently, since the 2005 edition of the NBC, use of the ESFP described in NBC Article 4.1.8.11. has been allowed for structures that meet any of the criteria specified in NBC Article 4.1.8.7.

30. For all other structures, dynamic analysis must be used. It is, however, recommended that all structures with a Type 9 irregularity be designed using dynamic analysis. For larger values of $I_E F_a S_a(0.2)$, the analysis should be non-linear dynamic and should include vertical earthquake motions.

Irregularities

31. The NBC 2005 and NBC 2010 included definitions of eight types of irregularities and specifications regarding analysis and design for each of those types. The Type 1 to 8 irregularities are retained in the NBC 2015, and a new Type 9 irregularity has been introduced for application to the design of structures that are considered to experience gravity-induced lateral demand on their SFRS. The kinds of specifications applicable to the different types of irregularities include the following: limitations on the use of the static analysis procedure, restrictions on irregularities permitted in relation to the extent of seismic hazard, restrictions applicable to post-disaster buildings, increases in seismic design forces, and specific design requirements (e.g., related to diaphragms, openings, and discontinuities).

A detailed description of the rationale behind the provisions on irregularities of Types 1 to 8 is given by DeVall.[10] The NBC 2015 continues to have specific requirements for taking torsional effects into account; a torsional sensitivity parameter, B, is used to determine whether or not dynamic analysis is required. The basis for these torsional design requirements is given by Humar et al.[11]

Dynamic Analysis Requirements

32. Dynamic analysis plays a prominent role in the NBC 2005, NBC 2010 and NBC 2015 seismic provisions, the general rationale being that Linear Dynamic Analysis—particularly the Modal Response Spectrum Method—is a straightforward procedure that simulates the effects of earthquakes on a structure much better than the ESFP. The Dynamic Analysis Procedure is the required method of analysis, except that the ESFP may be used for the following structures:
(a) structures in areas of relatively low seismicity, as defined by a short-period, importance-modified design spectral response acceleration, $I_EF_aS_a(0.2)$, of less than 0.35,
(b) regular structures less than 60 m in height with a fundamental lateral period of less than 2 s, and
(c) certain irregular structures less than 20 m in height with a fundamental lateral period of less than 0.5 s.

These exceptions recognize that:
(a) there is not likely to be any significant negative consequence to allowing the use of the ESFP in areas of low seismicity,
(b) the equivalent static loads can simulate dynamic effects for medium-height regular structures provided that the fundamental lateral period is not too long, and
(c) both overall force and distributional effects are determined quite well by the ESFP for relatively squat, short-period, irregular structures, except for those that are torsionally sensitive.

33. Conducting dynamic analysis in accordance with the NBC 2015 provisions is facilitated by the fact that seismic hazard is specified in terms of spectral response acceleration. Design spectral response acceleration values are determined from 5% damped spectral response acceleration values, $S_a(T)$, multiplied by site coefficients, F(T), for periods, T, of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s. This means that the input to a dynamic analysis is based on the best available estimates of ground motion at the specified probability of exceedance. The NBC 2015 requires that the spectral response acceleration values used in the Modal Response Spectrum Method be the design spectral response acceleration values (which are also used as the basis for determining the minimum lateral earthquake design force, V, in the ESFP) and that the ground motion histories used in the Numerical Integration Linear Time History Method be compatible with a response spectrum constructed from the design spectral response acceleration values. Saatcioglu and Humar[12] discuss the different methods of dynamic analysis and the necessary considerations for modeling structures for such analysis.

34. Although dynamic analysis is currently the default procedure in the NBC, there is still concern that the resultant seismic forces may be too low because the parameters used in the analysis (e.g., structural stiffness) are entirely at the designer's discretion rather than being specified by the Code. For example, while there are limitations on the maximum value of the fundamental lateral period, T_a, that can be used in the ESFP, there are no such limitations in the specifications for the Dynamic Analysis Procedure. To guard against inappropriate choices of design parameters, the NBC 2015 requires that the dynamically determined lateral earthquake design force, V_d, be not less than 80% of the statically determined lateral earthquake design force, V, and that, in the cases of irregular structures for which dynamic analysis is compulsory and of buildings with more than 4 storeys of continuous wood construction, the minimum value of V_d be 100% of the statically determined V. In determining the minimum value of V_d, V can be calculated using the dynamically determined fundamental lateral period, T_a, provided that T_a is not larger than:
(a) for moment-resisting frames and single-storey buildings with flexible roof diaphragms of steel deck or wood, 1.5 times the period determined using the applicable empirical formula, and
(b) for braced frames and shear walls, 2.0 times the period determined using the applicable empirical formula.

Special Provisions

35. In the NBC 2010 and NBC 2015, many restrictions on structural systems are presented in NBC Table 4.1.8.9., which also specifies values of the ductility-related force modification factor, R_d, and the overstrength-related force modification factor, R_o, for each type of SFRS. The restrictions are governed by the importance-modified design spectral response acceleration determined at periods of 0.2 s and 1.0 s. NBC Table 4.1.8.9. allows the designer to immediately see the consequences of choosing a particular SFRS, both in terms of the factors R_d and R_o and any restrictions that may be applicable to a particular system.

36. The NBC 2015 provisions also contain additional restrictions, including some on structures with particular structural irregularities and some on foundation design requirements. The rationale for the different restrictions is discussed in more detail in the sections of this Commentary that deal directly with them.

37. The in-plane dynamic response of flexible roof diaphragms of steel deck or wood in single-storey buildings may affect the ductility demand imposed on the vertical elements of the SFRS and the force demands in the roof diaphragms. Provisions that account for these effects have been added to the NBC 2015.

Seismic Hazard

38. This section of the Commentary summarizes the main aspects associated with the determination of the seismic hazard values used in the NBC 2015. More detailed discussions are presented in Adams et al.[4] and in Atkinson and Adams,[13] as well as in the other references cited in this section.

Reasons for Recalculation of Seismic Hazard

Improved Seismicity Information

39. The 2005 and 2010 editions of the NBC relied on the 2003 seismic zoning maps developed between 1993 and 2003 using the catalogue of earthquakes acquired up to 1991. Many earthquakes have been recorded since then, and the additional seismicity data have improved the understanding of the geographical patterns of earthquake occurrence in many regions of Canada and the ability to estimate earthquake occurrence rates as a function of earthquake magnitude (Halchuk et al.[14] and Adams et al.[4]). Of note, large earthquakes have occurred in unexpected regions, such as Denali, Alaska, in 2002, and Haida Gwaii, British Columbia, in 2012.

Improved Understanding of Seismotectonics and Its Relationship to Seismic Hazard

40. Discoveries since the late 1990s have led to an improved understanding of the relationship between earthquake occurrence and the geological structure of the Earth's crust:
- (a) evidence of crustal deformation from GPS measurements indicating the steady and unsteady movement of large blocks of western Canada, which has implications for the recurrence rates of large earthquakes at the block boundaries;
- (b) evidence of episodic tremor (on seismographs) and slip (from GPS monitoring networks) in the Cascadia subduction zone off British Columbia, Washington and Oregon, which have implications for the activity of subduction zones with no evidence of great earthquakes, like the Explorer zone (Rogers et al.[15]); and
- (c) a revised hypothesis concerning the occurrence in eastern North America of larger earthquakes and their aftershocks in relation to the relatively young rift faults that compromise the integrity of the continental crust (Adams[16]), and additional investigation of the implications of such features for seismic hazard (Tuttle and Atkinson,[17] and Atkinson and Goda[18]).

Findings such as these have a significant influence on the determination of seismic hazard in western and eastern Canada.

Improved Estimates of Strong Seismic Ground Motion

41. Considerable research conducted since the early 1980s provided the spectral-response-acceleration-based ground motion relations (now often termed Ground Motion Prediction Equations or GMPEs) that formed the basis for the determination of seismic hazard values for the NBC 2005 and NBC 2010 (e.g., Atkinson and Boore,[19] Atkinson,[20][21] and Boore et al.[22]).

42. The last decade, in particular, has seen a significant improvement in the analysis methods used and an order-of-magnitude improvement in the amount of high-quality ground motion data, especially from large earthquakes on well-instrumented sites, like Chi-Chi (Taiwan, 1999) and Tohoku (Japan, 2011). Additional ground motion records and enhanced modeling have also significantly improved the GMPEs for eastern Canada. As a result, more high-quality GMPEs are available, and there is greater confidence in the ground motions for larger events (which were previously derived by extrapolation). Nevertheless, the scatter within the ground motion observations (aleatory uncertainty) and the differences among the various ground motion relations (epistemic uncertainty) remain large. These uncertainties need to be quantified so that they can be accurately reflected in the estimation of mean hazard values (see Paragraph 43). The GMPEs for the NBC 2015 seismic hazard model are described in detail by Atkinson and Adams.[13] They allow seismic hazard to be determined in the form of uniform hazard spectra, which are plots of spectral response acceleration ordinates at different periods, each ordinate having the same probability of exceedance.

Improved Seismic Hazard Computation

43. The NBC 2015 seismic hazard model was developed by applying the Cornell–McGuire probabilistic method (Cornell[23]) to a set of fault and areal earthquake sources in Canada and adjacent regions (Halchuk et al.[14]). The model includes the treatment of uncertainty in all of the significant input parameters, such as seismicity rates, upper-bound magnitudes, focal depth, ground motion relations and source zone models (McGuire[24]). Two kinds of uncertainty can be distinguished:

(1) aleatory uncertainty arising from the physical variability that is inherent in the unpredictable nature of future events, and

(2) epistemic uncertainty arising from differences in modeling assumptions, unknown or partially known parameters, and extrapolation beyond the range of observed data.

Realistic values of the various uncertainties can therefore be used to compute the ground motions for a target probability of exceedance at a desired level of confidence, e.g., mean, median or median plus one standard deviation.

Brief Description of the Parameters Used in the NBC 2015 Seismic Hazard Model

Seismic Source Zones

44. The seismic source zones used as the basis for the seismic hazard data in the NBC 2005 and NBC 2010 consisted of two models, which were distinguished primarily as historical and regional models designated H and R, respectively. In contrast to the historically based source zones used in the H model, the R model used larger regional zones based on seismotectonic/geological considerations.

45. For the NBC 2015 seismic hazard model (Adams et al.[4]), updated historical and regional models were combined probabilistically for locations in northeastern Canada. For locations in southeastern Canada, an additional type of source—a hybrid between historical and regional—was added to updated historical and regional models, giving three models that were combined probabilistically. For locations in western Canada, a single set of source models was used, but with variations in the geometry of the sources as appropriate (e.g., in the closest approach of the Cascadia subduction zone to southwestern British Columbia). In all cases, the boundaries of the individual source zones were revised to reflect new information, and their seismicity parameters were recalculated.

46. The NBC 2005 seismic hazard model included one fault source—the Queen Charlotte fault—and a deterministic treatment of the Cascadia subduction zone. The probabilistic NBC 2015 seismic hazard model includes fault sources for low-angle subduction thrusts, an updated treatment of the offshore Queen Charlotte fault source, and five active fault sources of strike-slip or reverse type in the Alaska–Yukon region (a single geometry is used for the strike-slip fault sources). Low-angle subduction thrust faults of variable depths are used to model the Juan de Fuca, Explorer and Winona segments of the Cascadia subduction zone and the Haida Gwaii thrust. The uncertainty in the down-dip seismogenic potential of these subduction thrust faults is important for the prediction of onshore hazard and has been modeled.

47. Certain other faults in Alaska and Washington were not explicitly included in the 2015 model, as their contributions to seismic hazard in Canada are adequately represented by the areal sources used in the model.

48. The source zone models mentioned above are applicable to the more seismically active parts of Canada. However, about half of the Canadian land mass is tectonically stable and has too few earthquakes to reliably define seismic source zones (see Figure J-1). Because large earthquakes can occur anywhere in Canada—albeit rarely in the more tectonically stable, less seismically active regions—it is important to have reliable estimates of seismic hazard in these regions as well. Estimates of levels of seismic activity in these stable regions are based on considerations, which were updated from those by Adams and Halchuk[25] and used in the NBC 2010.

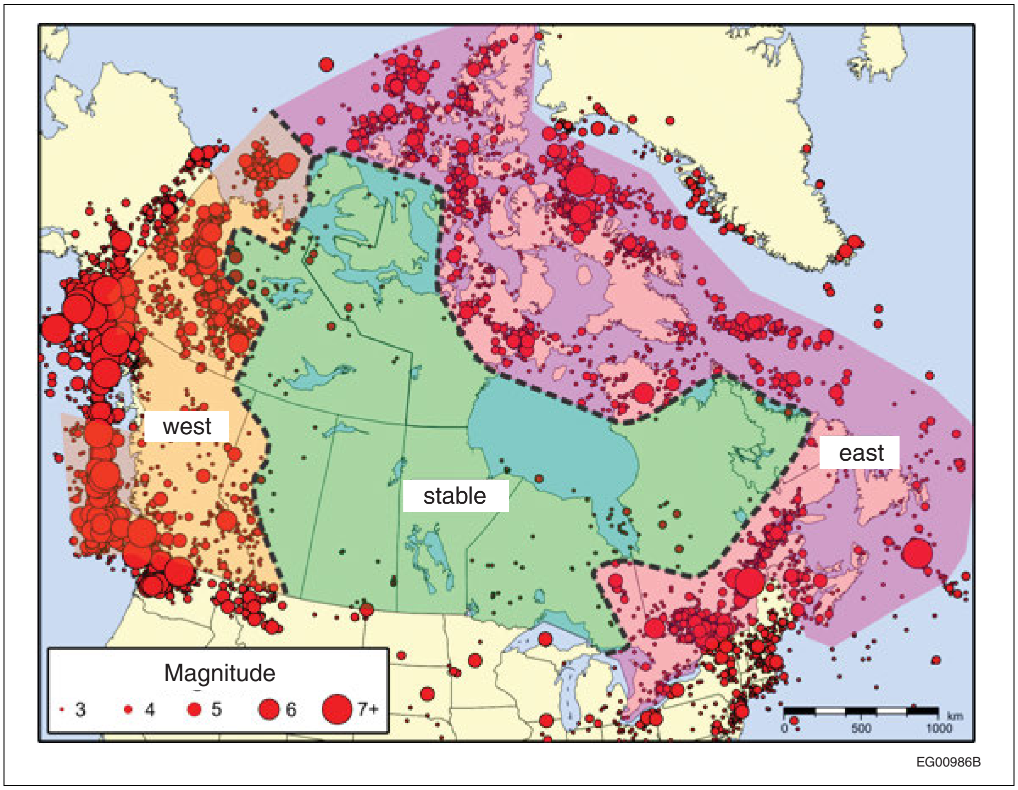


Figure J-1
Map of seismicity in Canada to 2010, delineating the eastern and western seismic regions and the low seismicity central continental region

Seismicity Parameters

49. The NBC 2015 seismic hazard model uses the data contained in the Canadian earthquake catalogue up to 2010. This catalogue contains a significant amount of additional data relative to the pre-1991 catalogue, which was used for the 2005 map, particularly for the Arctic. Older events in the catalogue have been revised (Bent[26]). The magnitudes of all earthquakes are now uniformly expressed on the moment magnitude scale; values on other magnitude scales were converted to moment magnitudes by using conversion equations as detailed in Halchuk et al.[27][28]

Magnitude-recurrence relations for the areal sources use an asymptotically truncated maximum-likelihood fit, with uncertainty included through the use of upper and lower curves that approximate 1.73 times the standard deviation. Earthquake recurrence models for the defined faults use information on the occurrence of prehistoric earthquakes or the number of earthquakes needed to accommodate the GPS strain rate. For the defined faults, both maximum-likelihood and approximately characteristic (Youngs and Coppersmith[29]) distribution models are used.

Upper-bound magnitudes for each source zone have been estimated for areal sources by considering the largest earthquakes observed in similar seismotectonic regions around the world and for defined faults by considering the seismogenic area of the fault. Earthquake depth is included in the estimates, even though the probabilistic seismic hazard for most of Canada is relatively insensitive to the exact depth used, the exception being southwestern British Columbia.

Ground Motion Prediction Equations

50. GMPEs are one of the most important components of the seismic hazard calculations, as they govern the amplitudes of ground motion estimated for any magnitude and distance. Although a common approach is to select a suite of appropriate GMPEs and use them (with their uncertainty) in a weighted combination as part of a full logic tree to compute seismic hazard values, this approach does not necessarily lead to a rational, consistent and transparent characterization of the uncertainty in expected ground motions across all magnitudes, distances and regions (Atkinson et al.[30]). Accordingly, Atkinson and Adams[13] identified a variety of approaches to simplify the relations and their uncertainty into central, upper and lower relations for each type of GMPE. As a deliberately conservative move, the relative weight on the three relations used for the NBC 2015 was modified from that recommended by Atkinson and Adams[13] to place slightly more emphasis on the upper relation (see Adams et al.[4]).

51. The GMPEs used for the NBC 2015 were all expressed for a reference ground condition at the B/C Site Class boundary. The values computed using these GMPEs were adjusted by applying the conversion factors listed in Atkinson and Adams[13] to provide the seismic hazard for Site Class C (at $\bar{V}_{s30} \approx 450$ m/s). Spectral response acceleration values computed using the GMPEs represent the 5% damped pseudo-acceleration at the specified vibration period, for the geometric mean of two horizontal components of the pseudo-acceleration.

52. Different GMPEs are needed depending on the nature of the Earth's crust and the type of earthquake source. The NBC 2015 seismic hazard model uses five central relations, with upper and lower relations for each one, which capture the epistemic uncertainty:
(1) eastern crustal GMPEs are used for eastern Canada, the Prairies and the eastern Arctic;
(2) western crustal GMPEs are used for crustal earthquakes in western Canada and Alaska;
(3) an in-slab GMPE is used for in-slab earthquakes in southwestern British Columbia and in Alaska;
(4) a subduction interface GMPE is used for low-dip subduction thrust events in southwestern British Columbia (including the Cascadia subduction zone) and in Alaska; and
(5) an "offshore" variant of the western crustal GMPE is used for other crustal earthquakes in the oceanic plate off British Columbia.

53. For most locations in Canada, the changes to the GMPEs are the most significant reason for differences between the NBC 2010 and NBC 2015 seismic hazard values. However, for areas of western Canada affected by the Cascadia subduction source, the differences in the values are mainly due to the probabilistic addition of the Cascadia contributions. In general, for locations in eastern Canada, the estimates of long-period seismic hazard increased, while the estimates of short-period seismic hazard decreased. For locations in western Canada, long-period seismic hazard estimates increased significantly in areas affected by the Cascadia subduction zone.

54. In most places in Canada, a single type of earthquake source contributes all of the seismic hazard (e.g., crustal earthquakes), but in southwestern British Columbia, the seismic hazard results from crustal, in-slab and subduction interface sources. The relative contribution of each type of source to the total seismic hazard depends on the period considered, but in general most of the short-period hazard results from in-slab earthquakes and most of the long-period hazard results from subduction interface earthquakes. Further details are given in Adams et al.[4]

Probabilistic Approach for Cascadia Subduction Earthquakes

55. In prehistoric times, the Cascadia subduction zone repeatedly generated large earthquakes (likely magnitude 9) off Vancouver Island (Adams[31] and Goldfinger et al.[32]). These earthquakes had ground motions of much longer duration than expected from the ground motions of nearby crustal and sub-crustal earthquakes. Geological records indicate a mean recurrence interval of about 600 years with a standard deviation of about 170 years; the last large earthquake in that area occurred in 1700.

For the NBC 2005 and NBC 2010, the Geological Survey of Canada adopted a magnitude 8.2 Cascadia earthquake scenario to provide a deterministic, rather than probabilistic, estimate of the expected ground motions (Adams and Halchuk[25]). For the NBC 2015, the Geological Survey of Canada has adopted a magnitude 9.0 (with uncertainty bounds) Cascadia earthquake scenario and has included it probabilistically in the seismic hazard calculations.

56. The NBC 2005 seismic hazard model treated just the Juan de Fuca segment of the Cascadia subduction zone, while the NBC 2015 seismic hazard model has additional contributions from the Explorer and Winona segments. As the evidence that the Winona segment is able to generate large earthquakes is equivocal, this segment is given a 50% chance of being seismogenic. For all segments, the down-dip extent of the rupture is given special attention in the 2015 model, as this is the part of the rupture most likely to generate damaging ground shaking in urban areas. Further information on the probabilistic handling of the Cascadia subduction earthquakes can be found in Adams et al.[4] and Halchuk et al.[14] The effects of the long-duration Cascadia ground motions are addressed by Tremblay,[33] Tremblay and Atkinson,[34] and Goda et al.[35]

Calculated Seismic Hazard Results and Design Ground Motions

57. Complete probabilistic calculations of seismic hazard were done for Canada to define the uniform hazard spectra for the geometric mean of the horizontal components of the 5% damped spectral response acceleration for the DGM periods, together with PGA and PGV. The seismic hazard calculations were performed by the Geological Survey of Canada using the proprietary FRISK88 software. Halchuk et al.[14] contains the input parameters for the model that was used, and the calculations can be reproduced (with minor differences due to the use of different algorithms) using the open-source EqHaz software available at www.seismotoolbox.ca. Thus, it is possible for knowledgeable users to reproduce and explore the calculations.

Choice of Confidence Level

58. The seismic hazard results presented in the NBC 2015 are mean hazard estimates of ground motion, including the model uncertainties. Those presented in the NBC 2005 and NBC 2010 were hybrid values obtained by taking the largest of alternative median hazard estimates from two or more seismotectonic models (the median hazard was preferred because it was considered more stable to changes in quantifying the uncertainty, which was in its infancy when the early models were developed). In related publications on the subject, the Geological Survey of Canada presented seismic hazard results for two confidence levels, the 50th percentile (median) and the 84th percentile, which include a measure of epistemic uncertainty. Although the 84th percentile is on occasion referred to as the median plus one standard deviation, this description is not entirely valid when dealing with seismic hazard because it is only applicable to symmetrical normal or lognormal distributions. Owing to the asymmetric nature of epistemic uncertainty, the distributions of ground motions about the median are quite asymmetric. Mean hazard values from probabilistic analyses are typically higher than the corresponding median hazard values because of the lognormal distribution of some of the variables in the model. For Canadian seismic hazard, mean hazard values typically lie between the 65th and 80th percentiles of the distribution (Adams et al.[4]).

59. It should be noted that the DGM values currently being calculated by the US Geological Survey for the US codes are also mean hazard values; however, differences in modeling approaches, particularly in the quantification of uncertainty, are such that the relationship between the American and Canadian DGM values is neither consistent nor easy to quantify. Consequently, direct comparison of Canadian and American DGM values—e.g., those for areas at or near the Canada–US border—even when adjusted to the same Site Class, has limited value and should be treated with caution.

Choice of Reference Ground Condition

60. It is essential that mapped values of seismic hazard be specified for the same reference ground condition for all of Canada to ensure that the hazard values are comparable in different parts of the country, even when computed using different source zone models and GMPEs.

61. The reference ground condition chosen for the seismic hazard values in the NBC 2010 was very dense soil and soft rock, i.e., Site Class C, with a time-averaged shear wave velocity in the upper 30 m of soil or rock, $\bar{V}_{s30}$, between 360 m/s and 760 m/s (see Table 4.1.8.4.A. in the NBC 2010). A backward analysis of this reference ground condition in light of the Atkinson and Boore[36] western crustal GMPEs indicates that the chosen condition had a $\bar{V}_{s30}$ of about 450 m/s, which is slightly lower than the midpoint of the Site Class C range. Site Class C is retained as the reference ground condition for the NBC 2015, but is explicitly defined as having $\bar{V}_{s30}$ = 450 m/s.

62. One advantage of choosing Site Class C as the reference ground condition is that it closely corresponds to the ground conditions used in determining the strong ground motion relations that are used in western Canada.

Seismic Hazard Values and Maps

63. Adams et al.[4] show seismic hazard results for a number of locations throughout Canada. NBC Table C-3 in Appendix C, which is based on information provided in their publication, shows mean hazard values at a probability of exceedance of 2% in 50 years (i.e., DGM values) of spectral response acceleration, $S_a(T)$, at periods, T, of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s, as well as of PGA and PGV, for the geometric mean of the horizontal components of the ground motions. These seismic hazard values, as well as those for locations not listed in NBC Table C-3, can be found by specifying the latitude and longitude of a particular location in the "Hazard Calculator" on the Earthquakes Canada Web site of the Geological Survey of Canada. This Web site also contains maps of $S_a(T)$, PGA and PGV. (See also Paragraph 93.)

64. Figure J-2 is a map of 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, for all of Canada. Figures J-3 to J-6 are maps of 5% damped spectral response acceleration at periods of 0.2 s, $S_a(0.2)$, and 1.0 s, $S_a(1.0)$, for southwestern and southeastern Canada, which are heavily populated regions having a significant seismic hazard. All $S_a(T)$ values are for Site Class C, i.e., very dense soil and soft rock. The purpose of providing these maps is to show how the pattern of seismic hazard varies in different geographical regions; they should not be used to obtain $S_a(T)$ values for specific locations.

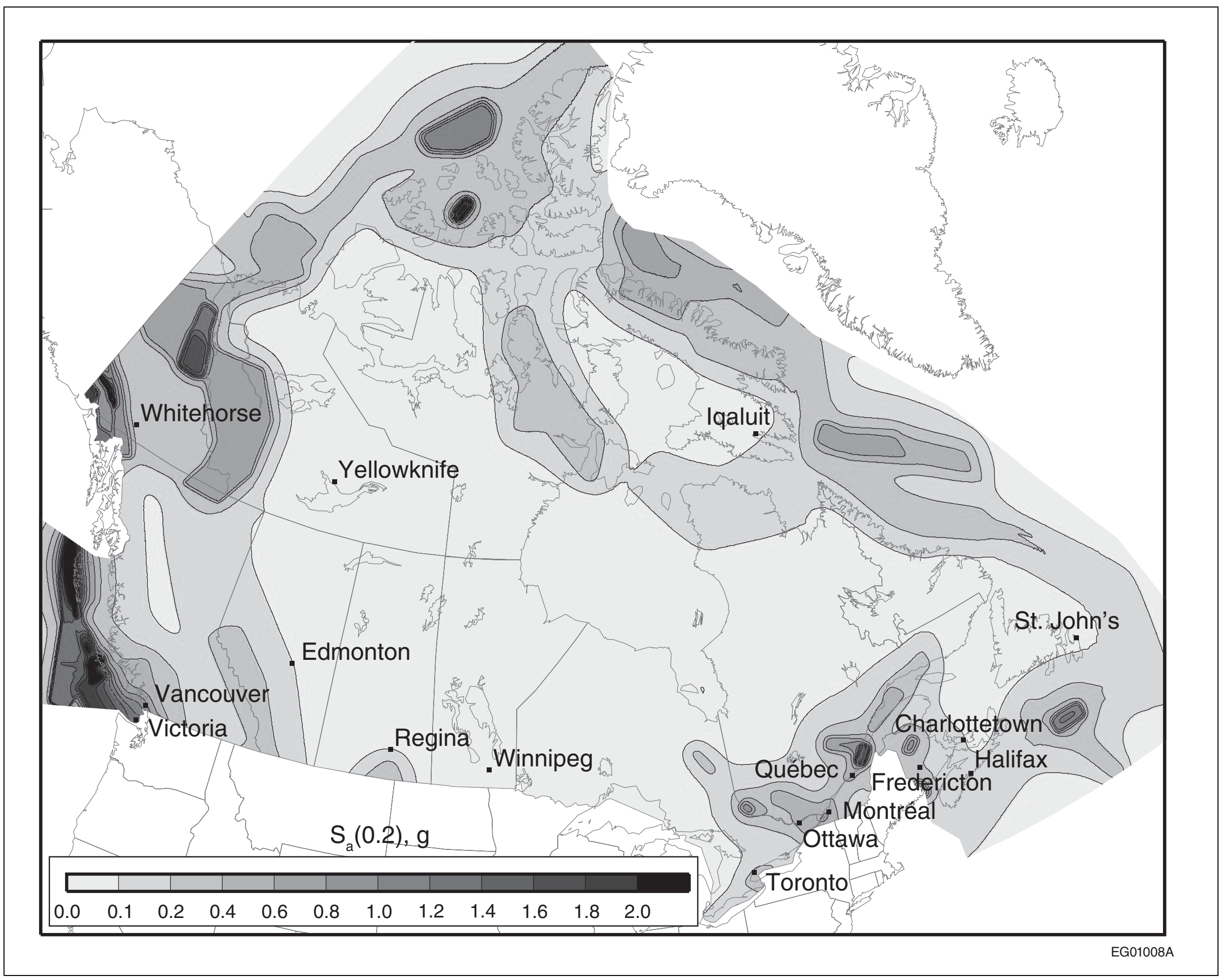


Figure J-2
Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, at a probability of exceedance of 2% in 50 years for Site Class C in Canada

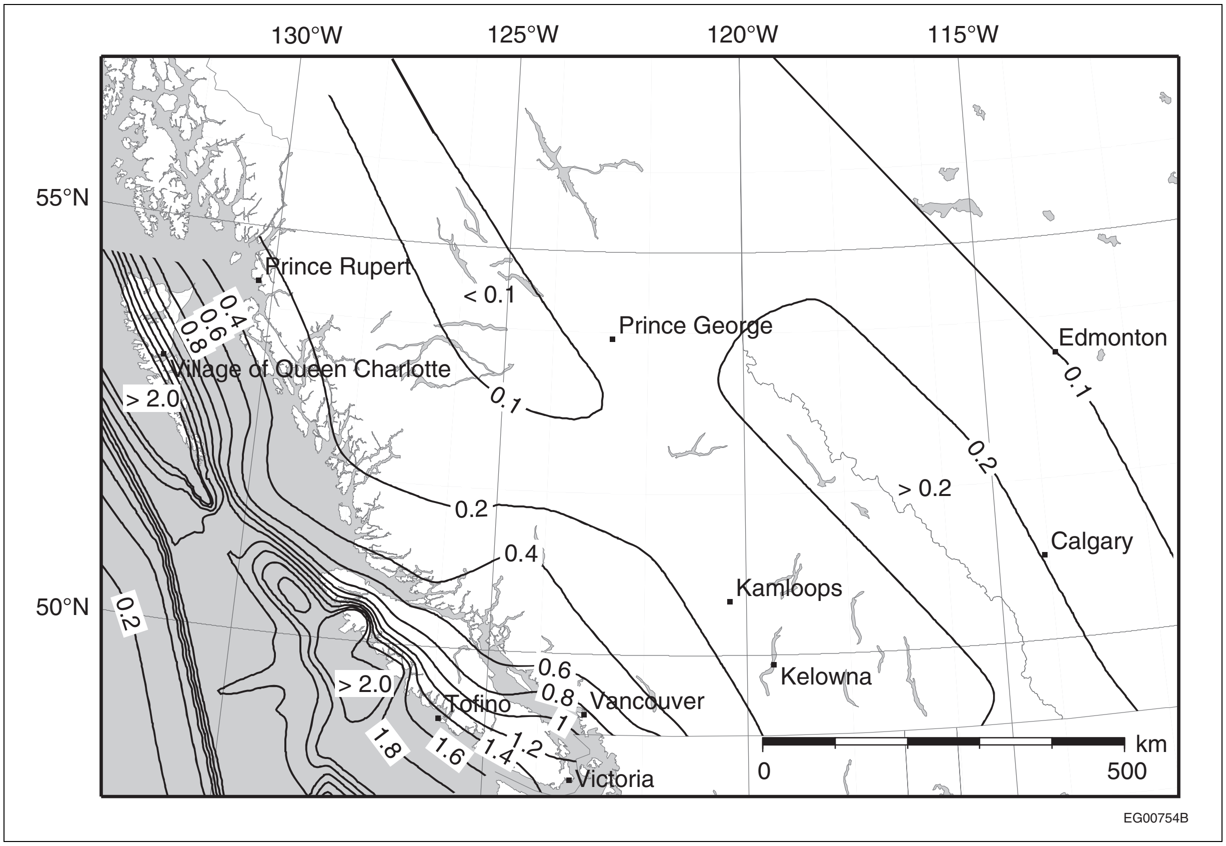


Figure J-3

Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in southwestern Canada

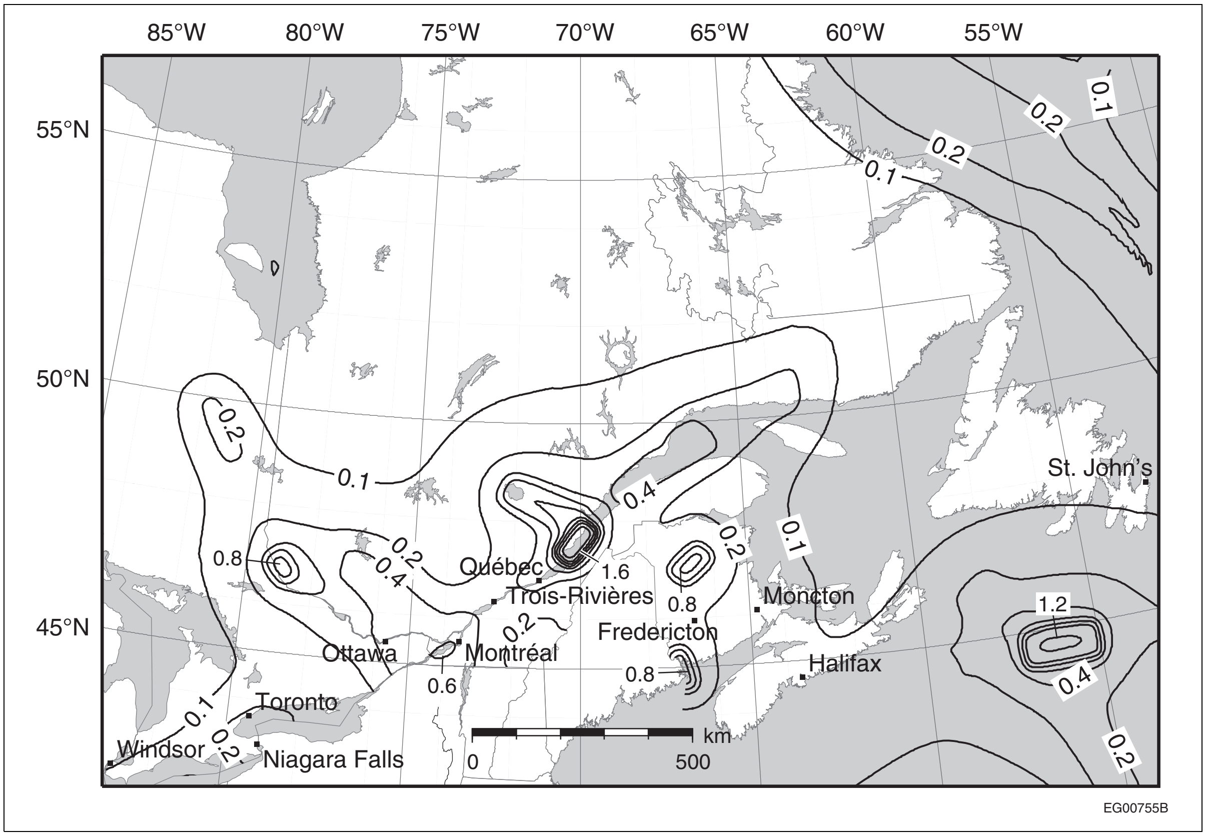


Figure J-4

Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in southeastern Canada

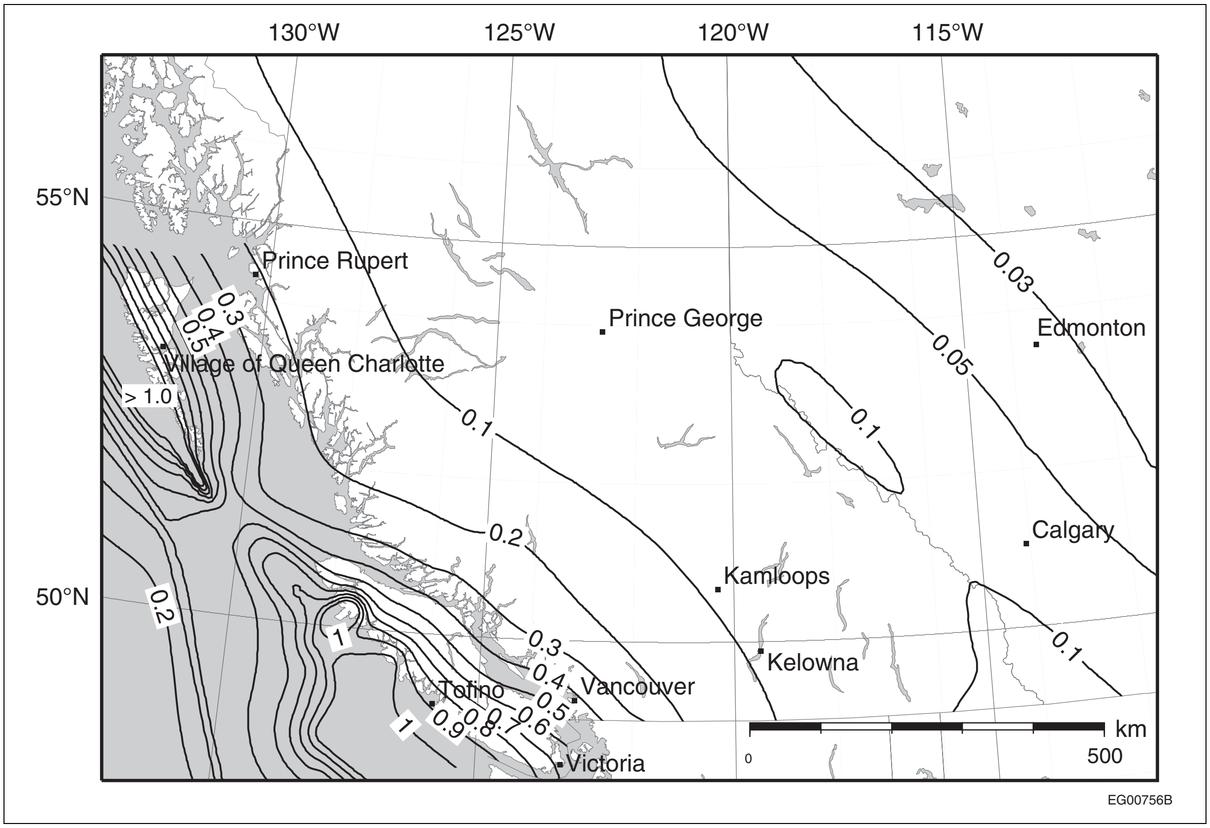


Figure J-5
Map of mean 5% damped spectral response acceleration at a period of 1.0 s, $S_a(1.0)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in southwestern Canada

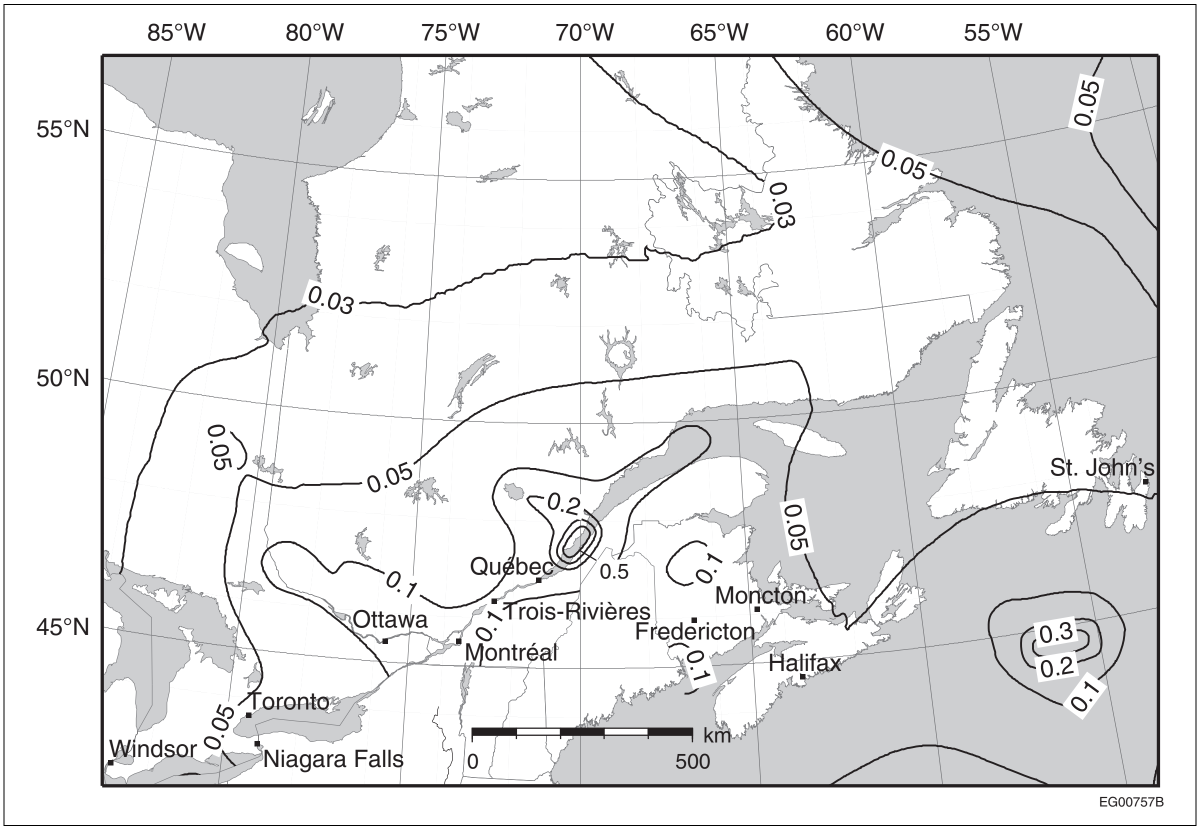


Figure J-6
Map of mean 5% damped spectral response acceleration at a period of 1.0 s, $S_a(1.0)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in southeastern Canada

Site Response Effects

Site Amplification

65. Site conditions play a major role in establishing the damage potential of incoming seismic waves from large earthquakes. Damage patterns in Mexico City caused by the 1985 Michoacan earthquake conclusively demonstrated the significant effects of local site conditions on the seismic response of the ground (Seed[37]). Peak accelerations of incoming motions in rock were generally less than 0.04 g and had predominant periods of around 2 s. Many clay sites in the dried lake bed on which the original city was founded also had site periods of around 2 s and were excited into resonant response by the incoming motions. As a result, the bedrock outcrop motions were amplified about 5 times. The amplified motions had devastating effects on structures with periods close to site periods. In the 1989 Loma Prieta earthquake, major damage occurred on soft soil sites in the San Francisco–Oakland region where the spectral response accelerations were amplified 2 to 4 times over adjacent rock sites (Housner[38]). It is clear that seismic design should incorporate the amplification effects of local soil conditions; this must be done effectively without unduly complicating the structural design process. Site coefficients (site amplification or foundation factors) are the preferred means used in seismic codes to capture the amplification effects of local soil conditions on ground motions and, hence, on seismic design forces.

Theoretical Basis of Site Amplification

66. The effects of site conditions on seismic ground motions usually refer to how seismic waves from the underlying rock are affected by the geometrical and geological structures of the softer surface deposits during wave transmission to the surface. An elementary theory on wave propagation (i.e., how ground conditions affect the waves) and its application to site response for building code purposes is given by Finn and Wightman.[6] The basic mechanism of amplification is best illustrated by examining the effect of a damped elastic surface layer on an incoming harmonic wave of period T_w from bedrock. As shown in Figure J-7, the elastic layer of soil is characterized by a thickness, H, a shear wave velocity, V_{ss}, a density, ϱ_s, and a fundamental period of $T_s = 4H/V_{ss}$; the shear wave velocity and density in the bedrock are denoted by V_{sr} and ϱ_r, respectively.

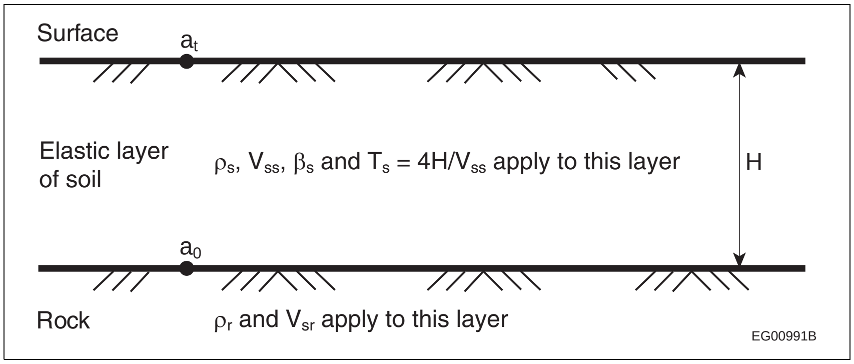


Figure J-7
Amplification in an elastic layer of soil on rock

67. At the time that the site amplification factors that formed the basis for the NBC 2005 and NBC 2010 site coefficients were developed, most strong motion instruments were located on rock or stiff soil sites. These instruments provided the database for predicting ground motions on such sites. Ground motions for seismic design on softer sites were determined by first estimating what the motions would be at the site on a rock or stiff soil outcrop and then estimating to what extent these motions would be amplified on passing through the soft overlying soils. The amplification ratio, A, between the outcrop acceleration, a_o, and the surface acceleration, a_t, when $T_w = T_s$ is calculated as follows:

$$A = 1/\left(\kappa + \beta_s\pi/2\right)$$

where β_s is the critical damping ratio, and $\kappa = \varrho_s V_{ss}/\varrho_r V_{sr}$ is the impedance ratio. The theoretical results so derived showed that the important parameters controlling ground motion amplification in elastic surface soil layers are:
(a) the relationship between the predominant period of the outcrop motions and the fundamental period of the surface layer,
(b) the impedance between the surface layer and the base material, and
(c) the damping in the surface layer.
Therefore, the key site parameters controlling the amplification of the outcrop motions are H, V_{ss}, κ and β_s.

Non-linear Site Amplification

68. Under strong shaking, the response of the soil will be non-linear. The shear modulus and damping are strain-dependent, and therefore the larger strains associated with strong shaking reduce the effective shear moduli and increase the damping. The shear strength of the soil also puts a limitation on the magnitude of the surface acceleration because the seismic waves cannot generate shear stresses greater than the mobilized shearing resistance of the soil. Field evidence shows that the non-linear behaviour of soils causes the ground motion amplification factors to be dependent on the intensity of shaking.

69. In Figure J-8, Idriss[39] has conveniently summarized the non-linear relationship between peak accelerations on soft soil sites and those on associated bedrock sites. The median curve is based on data recorded in Mexico City during the 1985 Michoacan earthquake and on strong motion data from the 1989 Loma Prieta earthquake. The part of the median curve for peak rock accelerations greater than 0.2 g is based on one-dimensional site response analyses using the SHAKE computer program (Schnabel et al.[40]). The curve suggests that, on average, the bedrock accelerations are amplified in soft soils until the peak rock accelerations reach about 0.4 g. The higher amplification ratios between rock and soil sites, in the range of 1.5 to 4, are associated with rock acceleration levels of less than 0.10 g when the response is closer to being elastic. The increased non-linearity of soft soil response at the higher accelerations reduces the amplification ratios because of the increase in hysteretic damping and the reduction in effective shear moduli.

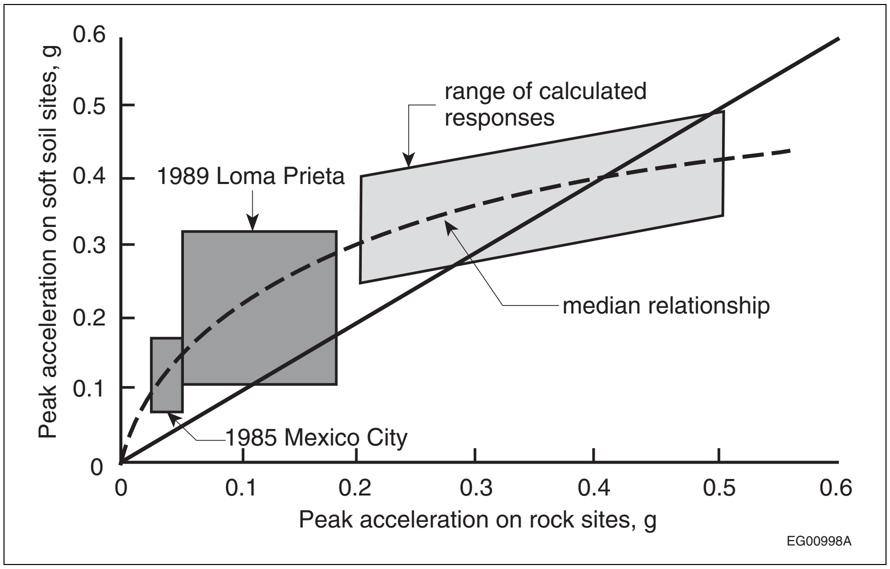


Figure J-8
Relationship between peak accelerations on soft soil sites and associated rock sites (after Idriss[39])

Site Coefficients

70. In many building codes, including the NBC 2015, the amplification effects of local ground conditions are represented by site coefficients. The variety of ground conditions is condensed into five distinct Site Classes, and an amplification factor—termed a site coefficient or foundation factor—is associated with each Site Class, input acceleration and period. One advantage of using broad and well-defined Site Classes is that rather distinct patterns of ground response are associated with each Site Class; however, one disadvantage is that it is sometimes difficult to decide which Site Class a complex ground condition should be assigned to.

71. There are two key elements to establishing a reliable site coefficient:
(1) the ground conditions must be quantitatively characterized into Site Classes, and
(2) a numerical amplification factor that is dependent on the frequency and intensity of shaking must be assigned to each Site Class.

In FEMA 222A, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions," NEHRP adapted the research of Borcherdt[41][42][43] and Dobry et al.,[44] and developed an approach using two site coefficients, F_a and F_v, to describe the amplification of outcrop motions in the short- and long-period ranges, respectively (see Figure J-9). NEHRP defined Site Classes primarily in terms of the average shear wave velocity in the top 30 m of the soil profile, $\bar{V}_{s30}$. To facilitate the use of these new Site Classes in practice, complementary descriptions based on standard penetration resistance and undrained shear strength were developed.

72. Values of F_a and F_v for each Site Class were specified for different levels of spectral ground acceleration. The values for F_a were mean values. The values of F_v derived from the research studies were highly variable, depending on site conditions and input ground motions; they were, therefore, given at the mean plus one standard deviation level.

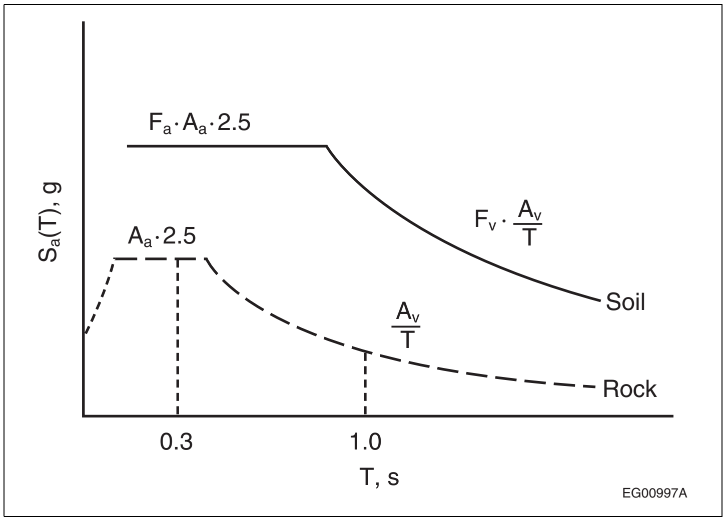


Figure J-9
Design response spectra based on period-dependent site coefficients (after FEMA 222A)

NBC Site Coefficients

73. The site classifications in the NBC 2015, as shown in NBC Table 4.1.8.4.-A, are identical to those in the NBC 2010. However, the short- and mid-period acceleration- and velocity-based site coefficients, F_a and F_v, respectively, presented in the NBC 2010 have been replaced in the NBC 2015 by period-dependent site coefficients for spectral acceleration, F(T), for fundamental periods, T, of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s; by site coefficients for PGA, F(PGA); and by site coefficients for PGV, F(PGV) (see NBC Tables 4.1.8.4.-B to -I).

74. The measure of ground motion intensity used to enter these tables has been changed from $S_a(0.2)$ and $S_a(1.0)$ in the NBC 2010 to an adjusted measure of PGA, PGA_{ref} (the reference PGA), in the NBC 2015. Because the attenuation of short-period ground motion is less in eastern Canada than it is in western Canada, the direct use of PGA would give F(T) values with larger non-linear de-amplification effects than appropriate for the sustained level of shaking at sites in eastern Canada. To avoid such unconservative F(T) values, which have potential safety implications, PGA_{ref} is defined in NBC Sentence 4.1.8.4.(4) as 0.8 PGA for sites where the ratio $S_a(0.2)$/PGA is less than 2.0. This adjustment ensures that appropriate site coefficients are used for sites in eastern Canada.

75. As previously mentioned in this Commentary, the reference ground condition used for the determination of seismic hazard in the NBC 2015 is Site Class C, which is very similar to that used in the NBC 1995. For all intensities of ground shaking, the site coefficient for Site C is 1.0. The site coefficients for Site Classes A, B, D and E were determined by maintaining the relative amplifications between each Site Class and Site Class C, as derived from the GMPE by Boore and Atkinson,[8] which take advantage of the growing dataset of ground motions recorded for various ground conditions. This approach, which has its origins in the work of Choi and Stewart,[7] has been adopted because the Boore and Atkinson GMPE forms the backbone of the crustal GMPE set used in the modeling of seismic hazard in western Canada (Atkinson and Adams[13]). There is thus an internal consistency between the site coefficients and the Site Class C reference ground condition seismic hazard values adopted for the NBC 2015.

76. The site coefficients for Site Classes A, B, D and E are determined relative to that for Site Class C (1.0) by using $\bar{V}_{s30}$ values that are characteristic of each Site Class. It was decided that these characteristic $\bar{V}_{s30}$ values would be determined not as the average of the $\bar{V}_{s30}$ range for each Site Class, as given in NBC Table 4.1.8.4.-A, but as the $\bar{V}_{s30}$ value that would yield a site coefficient that is an average of the range of site coefficients for each Site Class. The characteristic $\bar{V}_{s30}$ values that were used to calculate the site coefficients in the NBC 2015 are given in Table J-1.

Table J-1
Characteristic Values of $\overline{V}_{s30}$ for Site Coefficient Determination

Site Class	$\bar{V}_{s30}$, m/s
A	1 600
B	1 100
C	450
D	250
E	115

Seismic Design Not Included in NBC Provisions

Evaluation and Rehabilitation of Existing Buildings

77. Although the NBC seismic provisions are primarily intended for new buildings, they can also be used for the evaluation of the seismic adequacy of existing buildings. Commentary L contains general considerations for the structural evaluation and upgrading of existing buildings, as well as some discussion on earthquake considerations, including difficulties in applying the NBC seismic provisions for this purpose.

78. ASCE/SEI 41, "Seismic Evaluation and Retrofit of Existing Buildings," contains a comprehensive approach to the seismic retrofit of existing buildings. This standard presents a performance-based design approach and uses simplified deformation-based analysis procedures that explicitly recognize the non-linear behaviour of building components and elements; the standard also addresses the use of seismic isolation and energy dissipation devices.

NBC Subsection 4.1.8., Earthquake Load and Effects

Analysis (NBC Article 4.1.8.1.)

79. In the NBC 2005 and NBC 2010, NBC Article 4.1.8.1. specified that earthquake loading did not need to be considered in the design of buildings where $F_aS_a(0.2)$ was less than or equal to 0.12, but that where $F_aS_a(0.2)$ was greater than 0.12, all the requirements of NBC Subsection 4.1.8 had to be applied. In the NBC 2015, NBC Article 4.1.8.1. has been revised in view of the following:

(a) The exemption from designing for earthquake loading in regions of low seismicity implied that the seismic hazard in those regions was small compared to that in regions of higher seismicity, not that it was zero. In fact, for many building types in the exempted regions, the lateral earthquake design loads arising from the DGMs are much larger than the design wind loads.
(b) Seismological studies and earthquake data have indicated that earthquakes with a magnitude as large as 7 can occur anywhere, even in regions with little history of earthquake activity. Although large earthquakes may be rare events in such regions and the probabilistic DGM values for those regions may be low, the hazard is not zero.
(c) There is tremendous scatter in earthquake data, resulting in a large uncertainty in ground motion predictions.
(d) NBC Subsection 4.1.8. has become quite long and complex over the years as the understanding of earthquakes has improved owing to the availability of more data and advancements in knowledge. However, many of the requirements in Subsection 4.1.8. of the NBC 2015 only apply to buildings in regions of higher seismicity. Buildings in regions of low seismicity can be exempted from many of the requirements.
(e) An underlying principle of seismic design has always been to provide a complete load path that has good connectivity between its elements, in order to maintain sufficient building integrity during evacuation in an earthquake. Many standards, such as ASCE/SEI 7, "Minimum Design Loads for Buildings and Other Structures," require minimum levels of design earthquake forces and connectivity for structural integrity in regions of low seismicity.

80. In the NBC 2015, buildings in regions of low seismicity are no longer exempted from being designed for earthquake ground motions. The current NBC Article 4.1.8.1. introduces a minimum lateral earthquake design force and a self-contained simplified analysis procedure for regions of low seismicity where $I_EF_sS_a(0.2) < 0.16$ and $I_EF_sS_a(2.0) < 0.03$. For Normal Category buildings ($I_E = 1.0$) on Class C sites (F_s or $F_a = 1.0$), the $I_EF_sS_a(0.2)$ threshold of 0.16 in the NBC 2015 is higher than the $F_aS_a(0.2)$ threshold of 0.12 in the NBC 2010. As such, some buildings that previously had to be designed observing all the requirements of NBC Subsection 4.1.8. can now be analyzed using the simplified procedure.

81. The simplified analysis procedure was developed by taking the requirements in NBC Subsection 4.1.8. that only applied to regions of low seismicity, making some slightly conservative simplifications to the requirements and grouping all the simplified requirements in NBC Article 4.1.8.1. The resulting analysis procedure is similar to the one used for wind loads. Structures designed by this procedure are required to satisfy only the non-seismic requirements of the applicable design standards referenced in NBC Section 4.3.

General Requirements (NBC Article 4.1.8.3.)

NBC Sentence 4.1.8.3.(1)

82. This Sentence is included to ensure that designers use both the requirements of NBC Subsection 4.1.8. and those of the applicable design standards referenced in NBC Section 4.3. when developing both the concept of how the building structure will resist earthquake ground motions and the details of the seismic design of the building structure.

NBC Sentence 4.1.8.3.(2)

83. This Sentence introduces the concept of transferring earthquake-induced inertial forces to the supporting ground through clearly defined load paths. This concept entails designing the structure so that it incorporates a systematic approach for transferring inertial forces generated in the more massive portions of the building (e.g., floor slabs) to columns or walls that are continuous to the foundation of the structure. Where there are discontinuities in the load path, other provisions must be satisfied (e.g., NBC Article 4.1.8.15.) to ensure that these discontinuities do not become zones of weakness.

NBC Sentence 4.1.8.3.(3)

84. The designer of a building is required to clearly define the SFRS, which is the part of the overall structural system of the building that is intended to provide earthquake resistance by being the load path through which inertial forces are transferred to the ground. The SFRS must have the following two primary attributes:

(1) sufficient strength to transfer loads to the ground, and
(2) sufficient stiffness to keep lateral deformation within acceptable limits.

The SFRS will typically experience inelastic behaviour under design earthquake loading and must be designed to satisfy the seismic requirements in the applicable design standards referenced in NBC Section 4.3. Some elements of the building's structural system may not be part of the SFRS (e.g., slender perimeter columns); although these are not intended to resist earthquake loads, they will be affected by such loads and, as such, their design must take these effects into account, as required by NBC Sentence 4.1.8.3.(5).

NBC Sentence 4.1.8.3.(4)

85. This Sentence ensures that only the SFRS is counted on to resist the specified earthquake loads. Although there may be some implicit lateral-load-resisting capability in other structural components, none of the earthquake-induced loads can be assigned to such components, as they are only designed to maintain their vertical load-carrying capability and not to maintain lateral stiffness or capacity. For example, if the SFRS comprises a core wall system, no earthquake-induced loads should be assigned to perimeter columns, which are designed to carry gravity loads but which may have nominal lateral load resistance.

NBC Sentence 4.1.8.3.(5)

86. This Sentence requires that the behaviour of structural framing elements that are not part of the SFRS be investigated. When such elements are subjected to earthquake-induced deformations associated with the lateral deflections calculated in NBC Article 4.1.8.13., they must retain their integrity while supporting the gravity loads for which they were designed. The integrity of such elements is assured if they behave elastically; if their deformations are inelastic, their load-carrying capacity must not be at risk. For example, slender columns at the perimeter of a building whose SFRS comprises a core wall system must be investigated to demonstrate that they retain their capability of supporting their tributary dead and live loads while subjected to the lateral interstorey drift associated with the maximum expected earthquake-induced deflections.

NBC Sentence 4.1.8.3.(6)

87. Some stiff elements, such as concrete, masonry, brick or precast walls or panels, whether structural or non-structural, may not be intended to be part of the SFRS. However, if such elements are not adequately separated from other structural elements (not just those that are part of the SFRS), then they can have major effects on the behaviour of the building during an earthquake. First, they can significantly change the dynamic characteristics of the building structure (natural period and mode shapes) by stiffening it, which will normally increase the inertial forces in the building structure and possibly lead to its collapse. Second, these stiff elements will be subject to loads for which they were not designed, making them vulnerable to failure, particularly since they are often relatively brittle and not capable of undergoing earthquake-induced deformation without failing. Third, such stiff elements can cause the failure of structural elements of the building by inducing forces or displacements for which the structural elements are not designed. For example, a partial-height infill wall can cause severe damage to an adjacent column. NBC Sentence 4.1.8.3.(6) requires that stiff elements be separated from all structural elements in the building so that no interaction can take place or that they be specifically made part of the SFRS. Separation to prevent interaction requires that the gap between a stiff element and another structural element be greater than the maximum earthquake-induced deformation in that part of the structure. For example, an infill masonry wall would have to be separated from adjacent columns by at least the amount of the computed maximum interstorey drift. If the designer chooses to make a stiff element part of the SFRS, e.g., by connecting a precast exterior wall panel to perimeter columns, then all of the requirements of NBC Subsection 4.1.8. would be applicable to the analysis and design of that specific element. In particular, the effect of any stiff elements on the structural period and on the deflection of the structure when subjected to earthquake-induced inertial load would need to be taken into account by appropriate modeling (see NBC Sentence 4.1.8.3.(8)).

88. Gypsum wallboard and stucco walls are not required to be separated from the SFRS. The effect of gypsum wallboard on the SFRS in wood-frame buildings is addressed in the provisions of CSA O86, "Engineering Design in Wood" (background information for the CSA provisions is given by Ceccotti and Karacabeyli[45]).

NBC Sentence 4.1.8.3.(7)

89. This Sentence is concerned with the effects of structural and non-structural elements that are not considered part of the SFRS on the building's response during a seismic event. Although such elements often contribute stiffness to the building structure, they are not considered to contribute to the earthquake resistance of the structure. However, even though they are not considered part of the SFRS, their presence can contribute significantly to the overall behaviour of the building structure during an earthquake. NBC Sentence 4.1.8.3.(7) identifies three particular situations involving such elements, which must be accounted for in the design process:

Clause (a): The presence of these elements adds stiffness, which reduces the fundamental lateral period, T_a, of the structure; if the decrease is more than 15% of T_a, then the reduced period must be used in determining the design forces.

Clause (b): Because the behaviour of the structure may be affected by the presence of these elements, they must be considered in determining the irregularity of the structure (as described in NBC Table 4.1.8.6.). However, the additional stiffness contributed by such elements cannot be used to make an irregular structure regular or to reduce the effects of torsion. For example, the stiffness of a wall element or a gravity frame that is not part of the SFRS cannot be used to eliminate or decrease an eccentricity that is due to the SFRS alone. Consider the case of a building with an offset core whose SFRS comprises a braced frame or wall; a stiff gravity frame on the opposite face of the building cannot be used to reduce the torsional eccentricity unless it is made part of the SFRS and designed accordingly.

Clause (c): The inclusion of elements that are not considered part of the SFRS may have an adverse effect on the SFRS, for example, by changing the load path and causing some parts of the SFRS to be subject to higher forces and/or deformations than would otherwise be the case. The design of the SFRS must take such adverse effects into account. For example, the SFRS may consist of moment-resisting frames along the building perimeter in one lateral direction and a central wall in the other lateral direction. However, there may be a gravity frame along the building perimeter parallel to the central wall that has a column in common with the moment-resisting frame of the SFRS. Because of the frame action in the gravity frame, this column will be subject to axial forces, shears and moments during lateral deformation; these additional forces must be accounted for in the design of the column as part of the moment-resisting frame.

NBC Sentence 4.1.8.3.(8)

90. This Sentence requires that the structural modeling of the SFRS incorporate a realistic representation of the magnitude and spatial distribution of building mass and structural stiffness; it specifically requires that modeling include the effects of unseparated elements that are deemed not to be part of the SFRS as stated in NBC Sentence 4.1.8.3.(6). Such modeling is required for:
(a) the determination of lateral deflections, as specified in NBC Article 4.1.8.13.,
(b) the calculation of torsional sensitivity, as specified in NBC Sentence 4.1.8.11.(10), and
(c) the determination of the fundamental lateral period of the structure, as specified in NBC Clause 4.1.8.11.(3)(d).

91. The modeling for each of these purposes must be consistent, i.e., it must use the same assumptions regarding structural properties and behaviour.

92. The following modeling considerations are specifically identified as being important to take into account:

Clause (a): The effects of cracked sections must be modeled in determining the stiffness and strength of reinforced concrete and reinforced masonry elements. CSA A23.3, “Design of Concrete Structures,” specifies the stiffness reduction due to cracking, which depends on the kinds of loads carried by such elements.

Clause (b): Modeling must include the finite sizes of members and joints; a model that overlooks this feature can result in a significant underestimation of the stiffness of the structure. The extent of underestimation will depend on the type of structural framing system and the relationship between member sizes and span lengths. It is particularly important to include finite member and joint sizes when beams frame into shear walls; using a line representation of the shear wall and considering the beams to be joined at the shear wall centre line—rather than at the edge of the shear wall—will result in a structural model that is significantly more flexible than the actual structure.

Clause (c): The effects of the interaction of gravity loads with the displaced configuration of the structure will increase lateral displacements and moments throughout the structure; these additional moments reduce the structure’s capacity to resist lateral loads. These effects, which are commonly known as P-delta effects, can be particularly significant in ductile structures where displacements tend to increase during each incursion into the inelastic range.

P-delta effects have only a small influence on the response of buildings to seismic forces when the storey shear capacities exceed certain minimum values and the slopes of the storey shear–displacement curves, including P-delta effects, remain positive for the anticipated seismic displacements. When shear capacities fall below the minimum values and the slopes of the storey shear–displacement curves become negative, the displacements during earthquakes can become unacceptably large. Consequently, it is important that P-delta effects be modeled and taken into account if significant. Although considerable research has been done on how to take P-delta effects into account (e.g., Paulay and Priestley,[46] MacRae et al.,[47] Tremblay et al.,[48] Bernal,[49][50] Montgomery,[51] and Gupta and Krawlinker[52]), there is no widely accepted method to estimate seismically induced P-delta effects that takes inelastic deformation into account. The following procedure, which is similar to that recommended by Paulay and Priestley,[46] is recommended.

Earthquake-induced forces, shears, overturning moments and torsional moments calculated at each storey level are to be multiplied by an amplification factor of $(1 + \theta_x)$ to allow for P-delta effects, where θ_x is a stability factor at level x (the storey under consideration), which is calculated as follows:

$$\theta_x = \frac{\sum_{i=x}^{n} W_i}{R_o \sum_{i=x}^{n} F_i} \frac{\Delta_{mx}}{h_s}$$

where

$\sum_{i=x}^{n} F_i$ = design seismic shear force at level x, which is equal to the sum of the design lateral seismic forces acting at and above level x as determined in NBC Sentence 4.1.8.11.(7),

$\sum_{i=x}^{n} W_i$ = portion of the factored dead plus live load at and above level x,

Δ_{mx} = maximum inelastic interstorey deflection as defined in NBC Sentence 4.1.8.13.(3),

h_s = interstorey height,

R_o = overstrength-related force modification factor, and

$R_o \sum_{i=x}^{n} F_i$ = measure of the capacity at level x.

The amplification factor of (1 + θ_x) need not be applied to displacements.

As shown for a single-storey building in Figure J-10, the procedure recommended to allow for P-delta effects is equivalent to proportioning the structure at each level x to resist an increased seismic shear force, $\sum_{i=x}^{n} F_i^*$, calculated as follows:

$$\begin{aligned} \sum_{i=x}^{n} F_i^* &= R_o \sum_{i=x}^{n} F_i + \sum_{i=x}^{n} W_i \frac{\Delta_{mx}}{h_s} \\ &= R_o \sum_{i=x}^{n} F_i \left(1 + \theta_x\right) \end{aligned}$$

In calculating $\sum_{i=x}^{n} W_i$, the dead load factor and companion load factors given in Load Case 5 of NBC Table 4.1.3.2.-A should apply. The live load may be reduced for large tributary areas in accordance with NBC Article 4.1.5.8. The calculation of $\sum_{i=x}^{n} W_i$ provides an estimate of the actual gravity load acting at the storey under consideration at the time of an earthquake.

With the seismic shear capacities at each storey increased to allow for P-delta effects, the ability of the strengthened structure to absorb inelastic energy during an earthquake is also increased. The interstorey deflections of the strengthened structure should be about the same as the deflections of the original structure with the P-delta effects taken to be zero (see Figure J-10).

If the stability factor, θ_x, calculated as described above is less than about 0.10, then P-delta effects can often be ignored. When the stability factor is more than 0.40, the structure should be redesigned to guard against potential instabilities during extreme earthquakes.

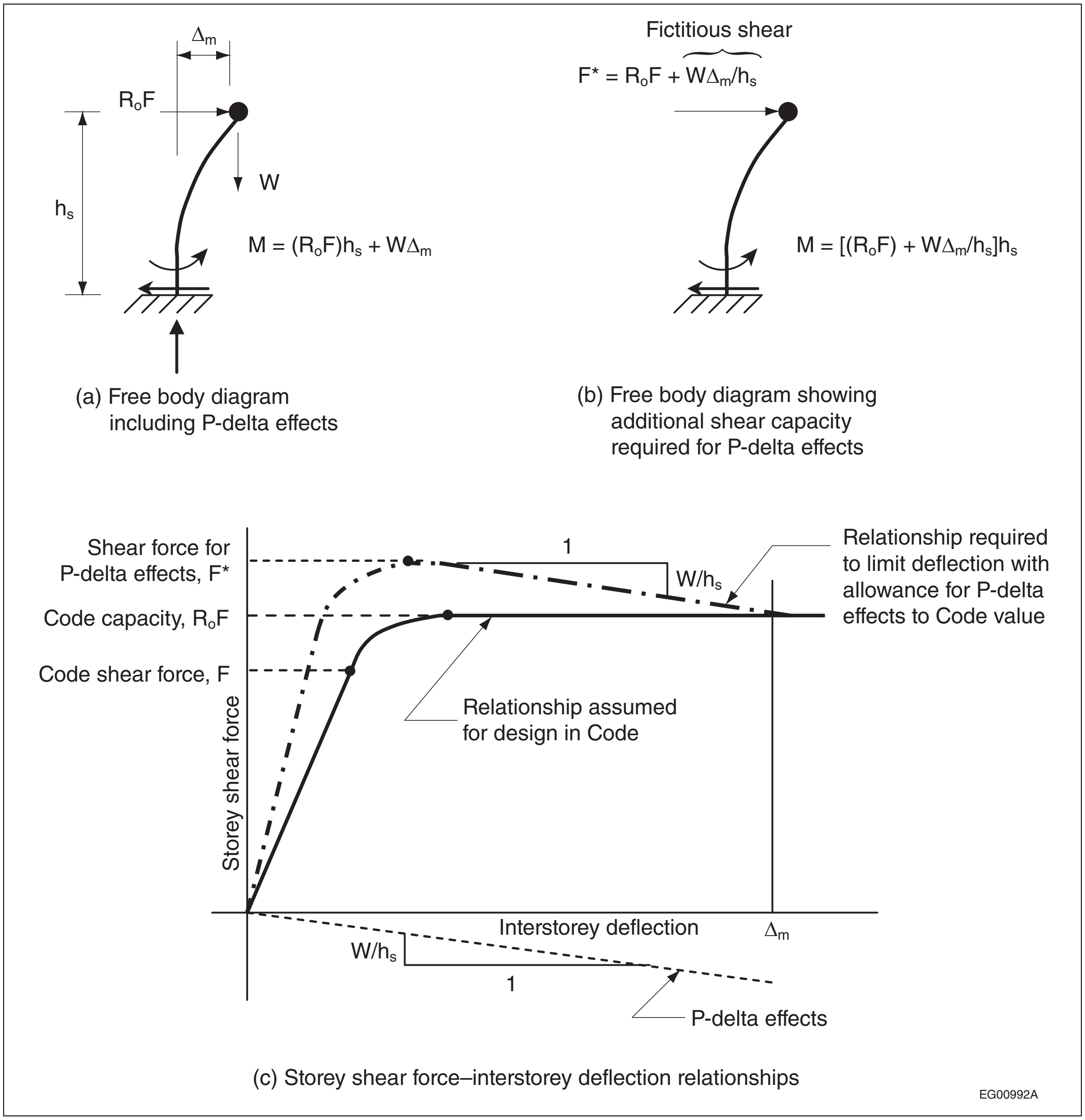


Figure J-10
P-delta effects in single-storey buildings

Although the method described above is conservative in most cases, it cannot guard against the risk of dynamic instability when large inelastic deformations are expected, particularly when ductility demand is concentrated in a few storeys. A static pushover analysis may be carried out to assess whether or not instability under P-delta effects is likely. This analysis must consider the impact of P-delta effects on the amplification of response, and the model used in the analysis must account for any degradation of strength in the structural elements as they undergo inelastic deformations. Compensation for P-delta effects is not necessary if the slope of the pushover curve remains positive for the anticipated seismic displacements. Humar et al.[53] have discussed the impact of P-delta effects and the pushover analysis used for determining the possibility of instability. FEMA P-695, “Quantification of Building Seismic Performance Factors,” provides additional information on non-linear static pushover analysis, including the impact of P-delta effects and strength degradation.

Clause (d): Modeling must take into account any other effects that might influence the lateral stiffness of the building, e.g., panel zone deformation in steel moment-resisting frames (Krawinkler et al.[54]). Lateral stiffness is a particularly important parameter for two reasons:

(1) the earthquake-induced load on the building is a direct function of the natural period, which itself is a direct function of lateral stiffness, and
(2) lateral stiffness is a major determinant of lateral displacement, which governs structural performance.

Site Properties (NBC Article 4.1.8.4.)

NBC Sentence 4.1.8.4.(1)

93. PGA, PGV and $S_a(T)$ values for the reference ground condition (Site Class C in NBC Table 4.1.8.4.-A) for many towns and cities in Canada can be found in NBC Table C-3 of Appendix C. PGA is used in the determination of site coefficients, and for liquefaction and other geotechnical analyses; PGV is not explicitly used in the NBC 2015, but is a useful parameter for predicting damage. Designers must use the design seismic hazard values for the location for which they are designing a building; values for locations not listed in NBC Table C-3 can be obtained from the Geological Survey of Canada (by using the "Hazard Calculator" on the Earthquakes Canada Web site) by specifying the applicable latitude and longitude. The methodology for determining these values, which are mean hazard results at a probability of exceedance of 2% in 50 years, is described in the Commentary section titled Seismic Hazard (starting at Paragraph 38). There can be significant gradients of the values of these ground motion parameters within urban regions, such as southwestern British Columbia, the western end of Lake Ontario, the Montréal region, and the Charlevoix region of Quebec. Figures J-11 to J-14 illustrate the variations of $S_a(0.2)$ in these four regions.

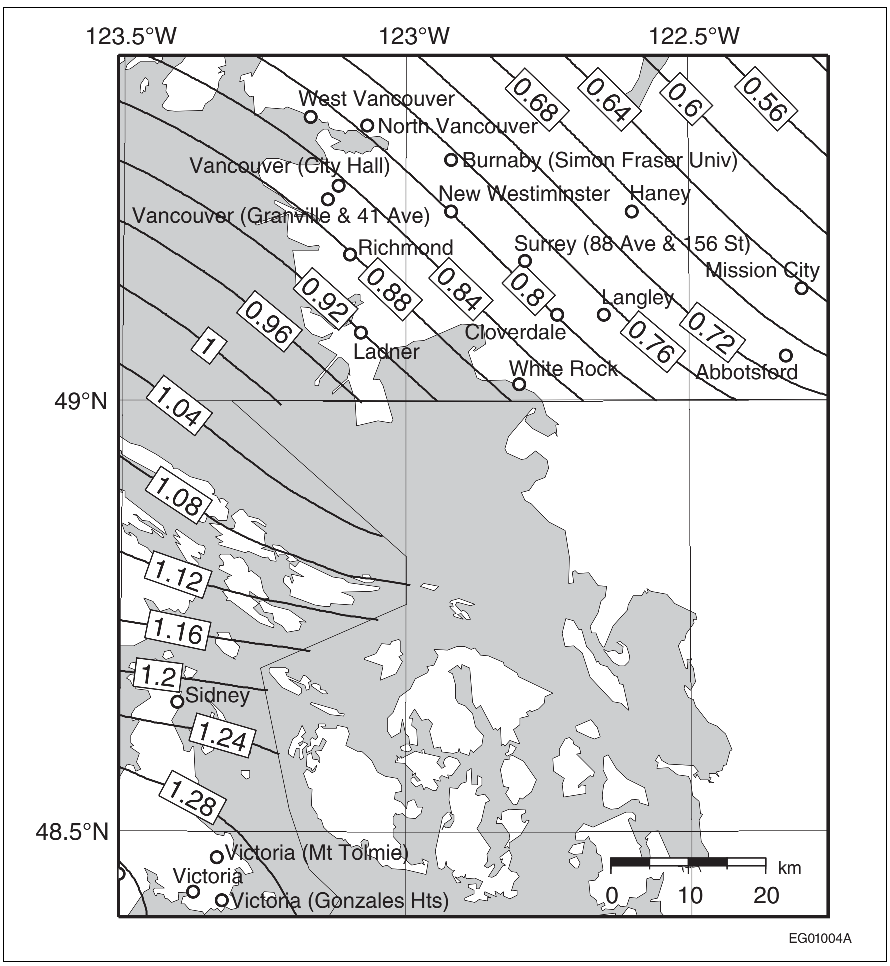


Figure J-11
Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in the Vancouver–Victoria region

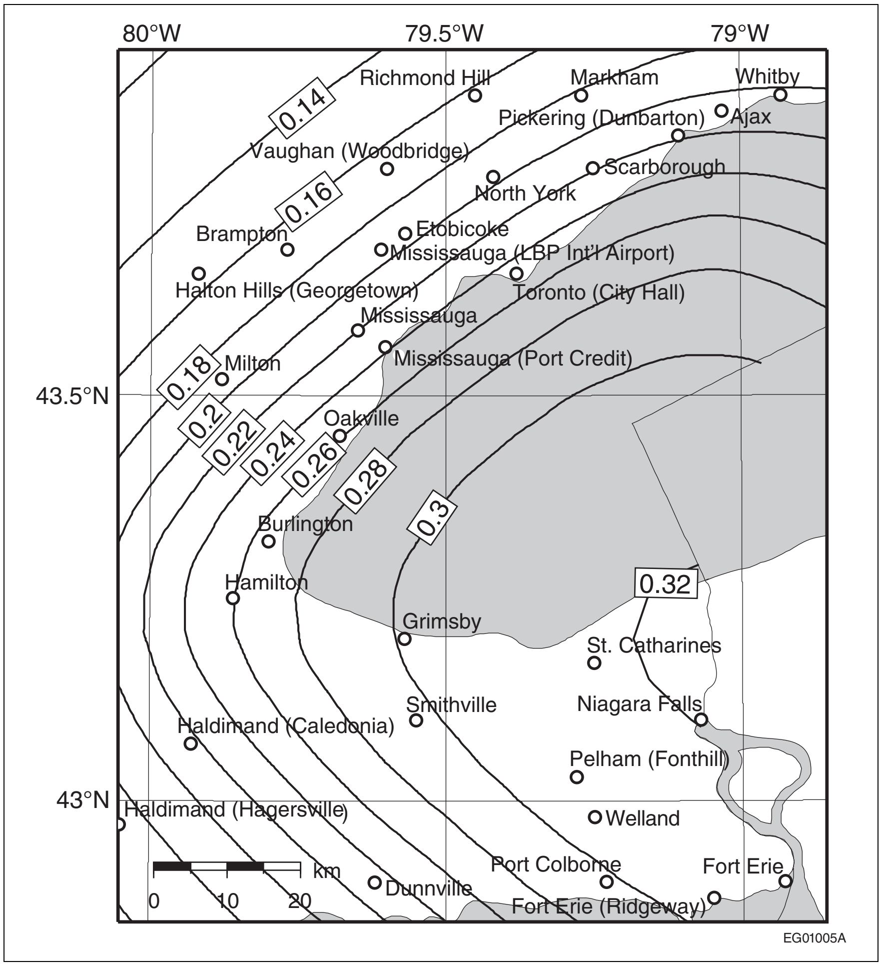


Figure J-12

Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in the Toronto–Niagara region

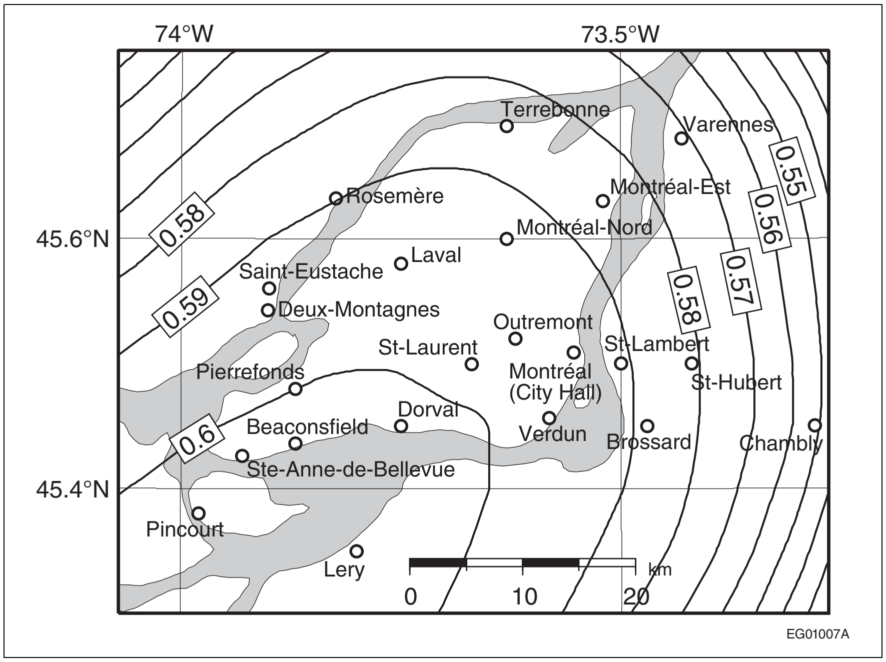


Figure J-13

Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in the Montréal region

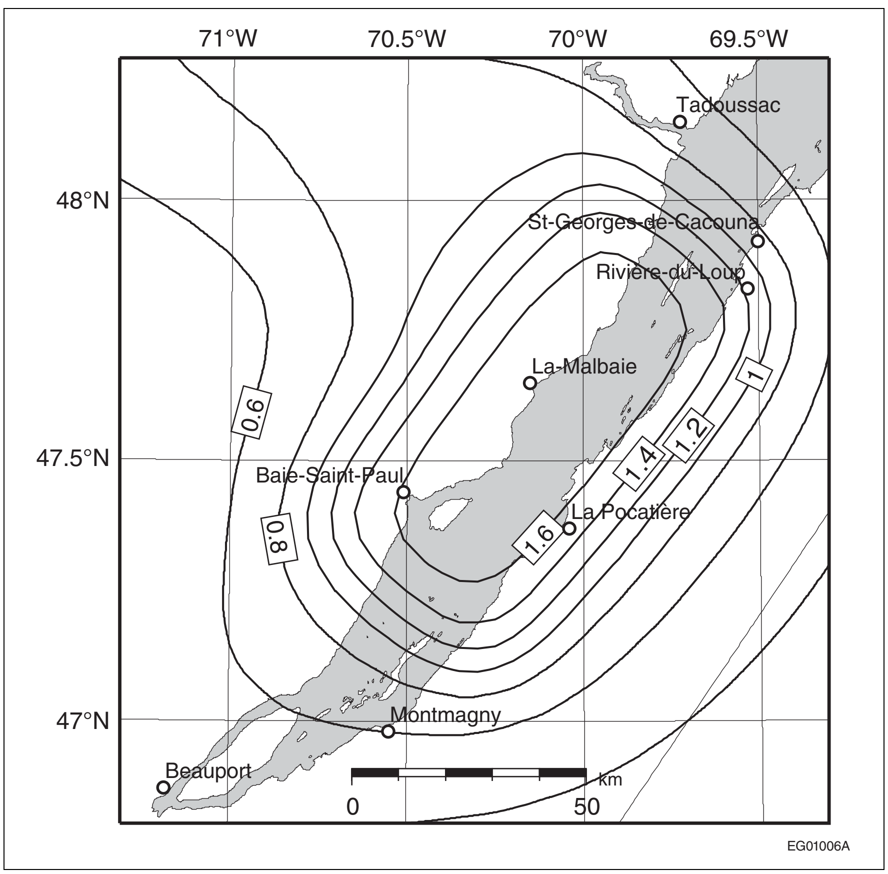


Figure J-14

Map of mean 5% damped spectral response acceleration at a period of 0.2 s, $S_a(0.2)$, in g, at a probability of exceedance of 2% in 50 years for Site Class C in the Charlevoix (Quebec) region

Design Guidance for Sites in the Yukon Close to Active Faults

94. The NBC seismic hazard values provide acceptable minimum standards for seismic design: they do not take into account the additional complexity of earthquake shaking near an active fault. The NBC 2015 seismic hazard model is the first to contain onshore fault sources, and some structures may be in close proximity to one of the modeled active faults. Coordinates for the fault sources are given in Halchuk et al.,[14] but are not precise enough to accurately locate a fault on or near a site under consideration. Therefore, detailed geological studies to determine the distance of the site from the fault and seismological studies to examine the near-fault ground motions at the site are recommended for sites within about 10 km of one of the modeled faults. There is a considerable body of expertise regarding the design of structures near active faults in California, and designers are encouraged to seek professional advice during the design process.

NBC Sentence 4.1.8.4.(2)

95. The rationale for using the site classifications given in NBC Table 4.1.8.4.-A is described in the Commentary section titled Site Response Effects (starting at Paragraph 65). The time-averaged shear wave velocity, $\bar{V}_{s30}$, is used to determine whether a rock site is of Class A or B. To distinguish between Site Classes C, D and E, it is preferable to use $\bar{V}_{s30}$, but where $\bar{V}_{s30}$ is not known, the energy-corrected average standard penetration resistance, $\bar{N}_{60}$, may be used for sand sites or the average undrained shear strength, s_u, for clay sites. Limiting values for $\bar{V}_{s30}$, $\bar{N}_{60}$ and s_u are given in NBC Table 4.1.8.4.-A. For the determination of $\bar{V}_{s30}$, $\bar{N}_{60}$ and s_u, the soil properties are averaged over a 30 m depth immediately beneath the bottom of the footings, pile caps or mat foundations supporting the SFRS. The allocation of a site to Class E is also based on the plasticity index, moisture content and undrained shear strength. By using this method of classifying sites, the difficulties associated with selecting a Site Class solely on the basis of a qualitative description, as was the case in pre-2005 editions of the NBC, are avoided. The adjectives “very dense,” “stiff” and “soft” used in NBC Table 4.1.8.4.-A are general descriptions and do not necessarily correspond to standard geotechnical usage, as defined in the Canadian Foundation Engineering Manual,[55] for example.

Buildings located on sloping bedrock sites or on soil profiles that are highly variable across the building footprint (e.g., if a portion of the foundation is on rock and the rest is on weak soil) and buildings with foundations of various depths require careful study, since the input ground motion may vary across the building footprint. In such cases, it may be necessary to carry out site-specific studies in at least two directions in order to evaluate the subsurface conditions and site-superstructure response. In some cases, the use of more than one Site Class may be warranted for portions of the same building with different foundation support. Where there are significant differences in site classification over the building plan area, a special study by qualified professionals is warranted.

All soil effects on every below-grade structural component of the building must be considered in the design of those components, with the input of a geotechnical engineer.

96. To classify a site as being in Site Class A or B, the shear wave velocity must be measured either on site or on profiles of the same rock in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m, shear wave velocity measurements at the surface may be extrapolated. If the site contains softer and more highly fractured and weathered rock than profiles known to be of Site Class B, either the shear wave velocity must be measured on site or the site must be classified in Site Class C.
The two rock categories—Site Classes A and B—are not to be used if there is more than 3 m of soil between the rock surface and the bottom of the spread footing, pile cap or mat foundation supporting the SFRS, even if the computed average shear wave velocity (see Paragraph 97) is greater than 760 m/s. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials.

The site coefficients, F(T), for Site Class A in NBC Tables 4.1.8.4.-B to -G are appropriate for buildings on hard rock where $\bar{V}_{s30}$ = 1 600 m/s. However, some rock sites, especially those on the Canadian Shield, have a $\bar{V}_{s30}$ in the range of 2 000 m/s to 3 000 m/s and will experience weaker shaking than a reference hard rock site. Note (2) to NBC Table 4.1.8.4.-A allows a relaxation of the F(T) values for buildings on hard rock when $\bar{V}_{s30}$ has been measured in situ; the adjustment given is a simple velocity calculation that does not take density changes into consideration, so a value of 1 500 m/s is used instead of 1 600 m/s.

97. Subject to the provisions of Paragraph 96, shear wave velocity can be determined using seismic cone, cross-hole or down-hole testing techniques (Kramer[56]) or using ambient vibration testing techniques (Sheri et al.[57] and Molnar et al.[58]). Where the 30 m consists of a number of distinctly different soil layers, the shear wave velocity for each layer must be determined and the average shear wave velocity, $\bar{V}_{s30}$, should be computed using the following equation:

$$\bar{V}_{s30} = \frac{\text{total thickness of all layers}}{\sum\left(\frac{\text{layer thickness}}{\text{layer shear wave velocity}}\right)}$$

NBC Sentence 4.1.8.4.(3)

98. Although it is preferable to determine the Site Class of non-rock sites on the basis of a measured $\bar{V}_{s30}$, it is permissible to use the energy-corrected average standard penetration resistance, $\bar{N}_{60}$, for sand sites or the average undrained shear strength, s_u, for clay sites (each averaged over the top 30 m of the site) when $\bar{V}_{s30}$ is not known. The alternative Site Class definitions in NBC Table 4.1.8.4.-A should not be used to infer any specific numerical correlation between the $\bar{V}_{s30}$, $\bar{N}_{60}$ and s_u values in the Table.

Where the 30 m is composed of a number of distinctly different soil layers, the standard penetration resistance or undrained shear strength for each layer should be averaged to provide $\bar{N}_{60}$ or s_u as follows:

$$\bar{N}_{60} = \frac{\text{total thickness of all sand layers}}{\sum\left(\frac{\text{layer thickness}}{\text{layer standard penetration resistance}}\right)}$$

$$s_u = \frac{\text{total thickness of all clay layers}}{\sum\left(\frac{\text{layer thickness}}{\text{layer undrained shear strength}}\right)}$$

Where the site classifications determined from $\bar{V}_{s30}$, $\bar{N}_{60}$ and s_u differ, the classification determined from $\bar{V}_{s30}$ governs. If only $\bar{N}_{60}$ and s_u are available, the one that gives the greater site coefficients governs.

The Site Class of a site with ground improvement is determined using the values of $\bar{V}_{s30}$, $\bar{N}_{60}$ and s_u for the improved site, averaged over a 30 m depth immediately below the bottom of the footings, pile caps or mat foundations supporting the SFRS.

99. If the site contains more than 3 m of soft soil (which is defined by a plasticity index of greater than 20, a moisture content of 40% or more, and an undrained shear strength of less than 25 kPa) under the footings, pile caps or mat foundations supporting the SFRS, then the site must be assessed as Site Class E, even if the averaged parameter, $\bar{V}_{s30}$, $\bar{N}_{60}$ or s_u, would otherwise classify it in a better Site Class. The rationale for this requirement is that soft soil layers as thin as 3 m can produce a large amplification of the underlying rock motion; this situation is somewhat analogous to the occurrence of large deflections resulting from the presence of a soft storey in a building structure.

100. Site Class F includes soils for which the determination of site amplification is problematic, such as liquefiable soils, highly sensitive clays, organic clays, highly plastic clays and thick layers of soft to medium-stiff clays. If the site has any of the four soil types described in Note (3) of NBC Table 4.1.8.4.-A, then it must be assessed as Site Class F, and the site coefficients, F(T), F(PGA) and F(PGV), must be determined by site-specific evaluation (including site response analyses), as specified in NBC Sentence 4.1.8.4.(6).

NBC Sentence 4.1.8.4.(5)

101. The site amplification approach described in the Commentary section titled Site Response Effects (starting at Paragraph 65) was used to derive the site coefficients, F(T), F(PGA) and F(PGV), specified in NBC Tables 4.1.8.4.-B to -I, which are used to modify the ground motion values so they are compatible with the different Site Classes. Values of F(T) are given for specific values of lateral time period as a function of the reference PGA, PGA_{ref}, in order to take into account the effect of non-linear soil response on site amplification. The variation of F(0.5) with PGA_{ref} for Site Classes A to E is shown in Figure J-15.

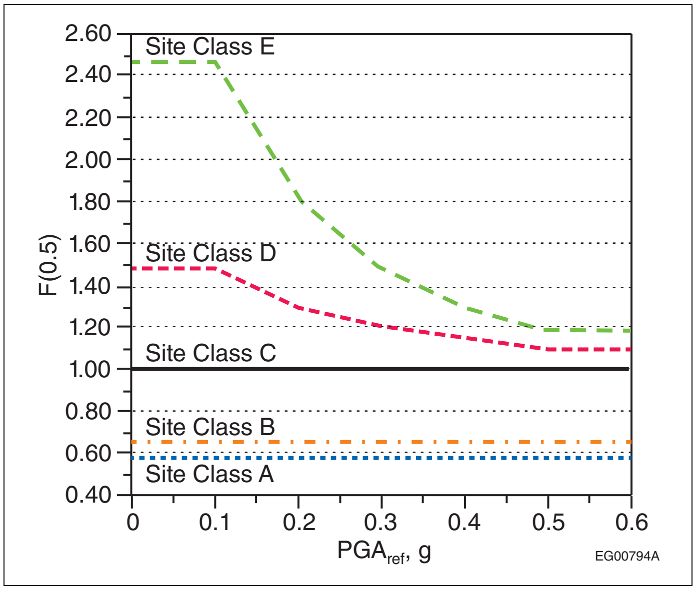
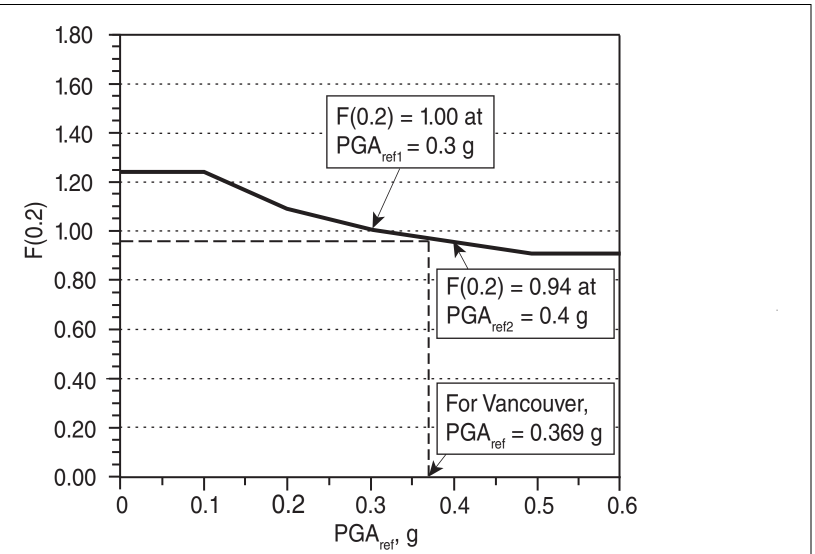


Figure J-15
Variation of F(0.5) with PGA_{ref} for Site Classes A to E

For a given Site Class, the values of F(T) for intermediate values of PGA_{ref} are obtained by linear interpolation between the values in the two relevant PGA_{ref} columns of NBC Tables 4.1.8.4.-B to -G (see Figure J-16).



$$F(T,PGA_{ref}) = F(T,PGA_{ref1}) + \left\{(PGA_{ref} - PGA_{ref1}) \times \left[\frac{(F(T,PGA_{ref2}) - F(T,PGA_{ref1}))}{(PGA_{ref2} - PGA_{ref1})}\right]\right\}$$

$$F(0.2,0.369) = 1.0 + \left\{(0.369 - 0.3) \times \left[\frac{(0.94 - 1.0)}{(0.4 - 0.3)}\right]\right\} = 0.96$$

EG00795A

Figure J-16
Linear interpolation to determine F(0.2) for Site Class D in Vancouver

NBC Sentence 4.1.8.4.(6)

102. Site-specific studies, including dynamic site response analyses, geotechnical investigations and evaluations, are required to determine the site response spectrum for Site Class F. Dynamic site response analyses require modeling of the soil profile, selecting input ground motions that are compatible with the response spectrum for the reference base material (rock or Site Class C), and conducting non-linear or equivalent linear dynamic analysis of the soil profile subjected to the selected input motions. A sufficient number of earthquake records—typically at least 11—should be used in order to accommodate the uncertainty associated with selecting input ground motions for analysis.

For soft clay sites, total stress analyses are acceptable. Sand sites only qualify as Site Class F if they are liquefiable. In such cases, site response analysis, if desired, should be conducted in terms of effective stress. In many cases, it will not be necessary to analyze the natural site if ground improvement is used to eliminate the potential for liquefaction through drastic altering of the site.

The average smoothed spectrum derived from the response spectra of 11 or more input ground motions used for site response analyses may be used as a design response spectrum (see Figure J-17). Since the response spectra directly reflect the effect of site conditions, there is no need to derive site coefficients.

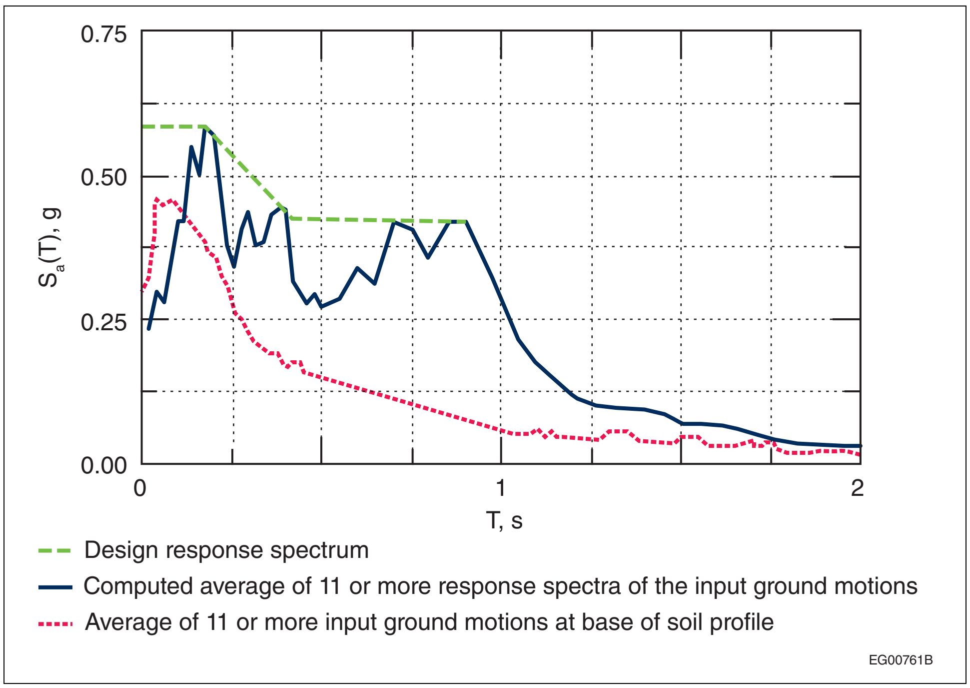


Figure J-17
Derivation of a site-specific design response spectrum for Site Class F from 11 or more site response analyses

NBC Sentence 4.1.8.4.(8)

103. For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, the values of F(T) can be determined from NBC Tables 4.1.8.4.-B and -C using the Site Class definitions in NBC Table 4.1.8.4.-A by assuming that liquefaction will not occur. This exception applies only for the purposes of defining the Site Class and obtaining site coefficients. The potential for liquefaction and its effects on structures as a ground failure hazard is nonetheless still required to be assessed.

NBC Sentence 4.1.8.4.(9)

104. This Sentence defines how the site coefficients for spectral acceleration, F(T), are used to modify the 5% damped spectral response acceleration, $S_a(T)$, to obtain the design spectral response acceleration, S(T), at T = 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10 s.

For some Site Classes in some locations, S(0.5) is larger than S(0.2). It is not considered good practice to design on the basis of a spectrum in which the S(T) value increases with the period. Because the period of a structure becomes longer as the structure responds in the inelastic range, a structure designed for a shorter period could migrate into a longer period range where it attracts higher forces. To avoid this possibility, S(0.2) is specified to be the greater of $F(0.2)S_a(0.2)$ and $F(0.5)S_a(0.5)$. For example, as shown in Figure J-18, for Site Classes D and E in Vancouver, S(0.5) is greater than S(0.2), and the short-period plateaus, indicated by dashed lines, in the design response spectra for these Site Classes extend from T = 0 to T = 0.5 s. The lower value of S(0.2) relative to S(0.5) is caused by non-linearity in the softer soils of these Site Classes. The short-period plateaus in the design response spectra for Site Classes A, B and C extend from T = 0 to T = 0.2 s, as usual.

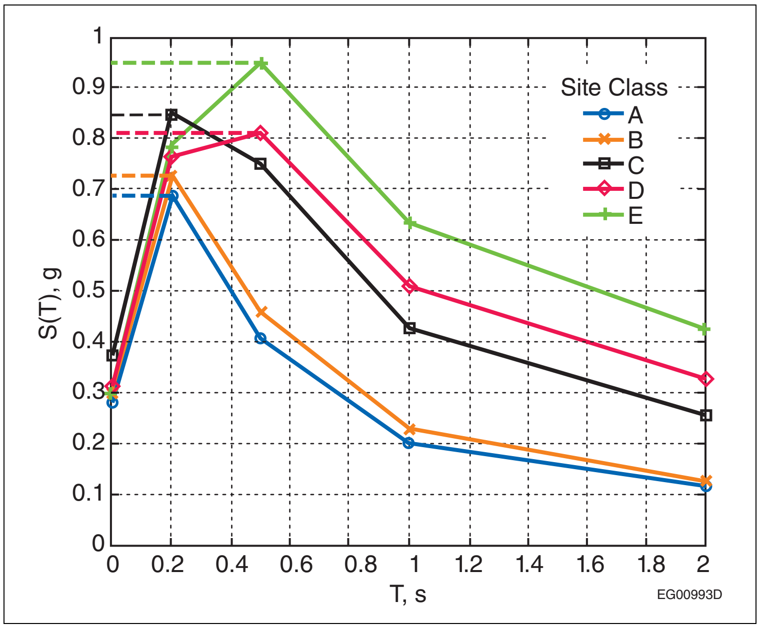


Figure J-18
Design response spectra for Site Classes A to E in Vancouver, where S(0.5) is greater than S(0.2) for Site Classes D and E

105. Stiffer soils generally amplify the ground motion less than softer soils. Thus, the site coefficients for Site Classes A and B are always less than 1, and the S(T) values for Site Classes A and B are lower than those for Site Class C at all periods (see the spectra for Vancouver in Figure J-18). However, softer soils may develop non-linearity in their response to strong levels of ground shaking, thus reducing the ground motion intensity. The net effect of the two phenomena may at times lead to a deamplification of the ground motion. As an example, for Vancouver, which has a PGA_{ref} of 0.369 g, the F(0.2) values for Site Classes D and E are 0.96 and 0.97, respectively, signifying deamplification relative to Site Class C. On the other hand, for the same two Site Classes, the F(0.5) values are 1.16 and 1.36, respectively. Accordingly, as shown in Figure J-18, the S(0.2) values for Site Classes D and E are lower than that for Site Class C, and the S(0.5) values for Site Classes D and E are higher than the S(0.2) values. The effect of non-linearity in softer soils is less significant for lower intensities of ground shaking. For example, as shown in Figure J-19, for Toronto, the S(T) values for Site Classes D and E are higher than those for Site Class C at all periods.

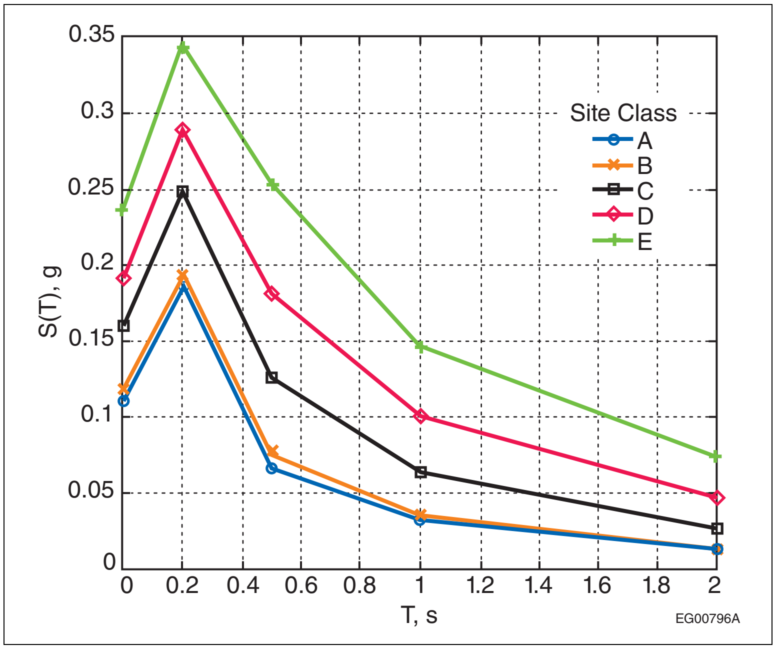


Figure J-19
Design response spectra for Site Classes A to E in Toronto, where softer ground conditions cause increased S(T) at all periods

106. Although spectral response accelerations, $S_a(T)$, at very short periods are typically about equal to or slightly less than $S_a(0.2)$, the design spectral response accelerations, S(T), for T < 0.2 s are specified to be equal to S(0.2). This degree of conservatism reflects the imprecision associated with the determination of periods for very stiff structures, since the period may well be somewhat longer than computed or damage to the structure may cause the period to lengthen and move into a higher response region. In some cases, $S_a(0.1)$ is, in fact, higher than $S_a(0.2)$, as shown in Figure J-20 for Niagara Falls, Toronto and Kingston. It would appear that, in such cases, a short-period structure could be under-designed by defining S(0.2) as the upper bound of the design spectral response acceleration.

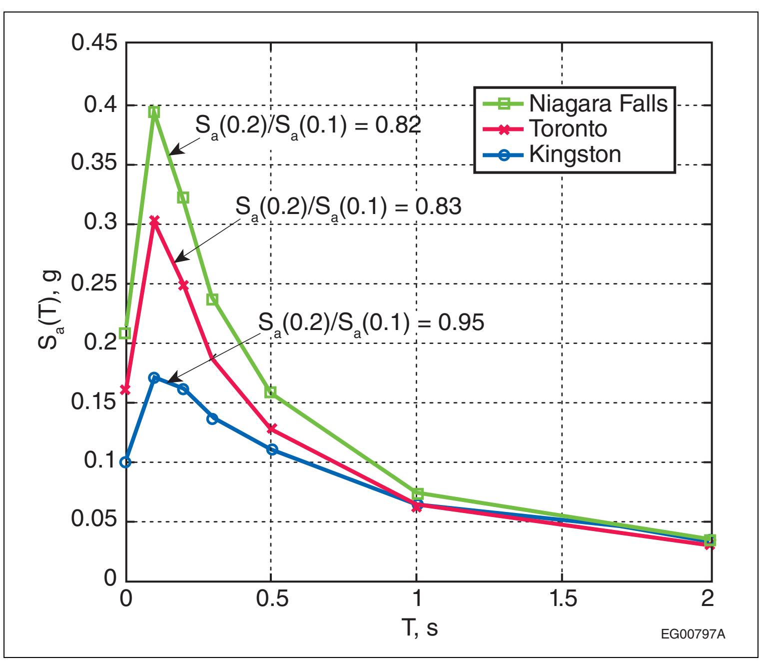


Figure J-20
Response spectra for Niagara Falls, Toronto and Kingston, where $S_a(0.1)$ is greater than $S_a(0.2)$

However, because S(0.2) is generally no less than about 80% of S(0.1), and because of the presence of significant reserve strength in short-period structures and the probable lengthening of the period, the upper plateau in the design response spectrum is still defined at S(0.2) in the NBC 2015. Although $S_a(0.1)$ is not specified in the NBC 2015, its value can be obtained from the Geological Survey of Canada (by using the "Hazard Calculator" on the Earthquakes Canada Web site).

Importance Factor (NBC Article 4.1.8.5.)

NBC Sentence 4.1.8.5.(1)

107. NBC Table 4.1.2.1. identifies four Importance Categories for buildings, which are based on intended use and occupancy: Low, Normal, High and Post-disaster. NBC Table 4.1.8.5. provides values of the importance factor for earthquake loads and effects, I_E, for each of these four categories for use in ultimate limit states design: 0.8 for Low, 1.0 for Normal, 1.3 for High and 1.5 for Post-disaster.

108. The earthquake importance factor, I_E, is primarily used to reduce the ductility demand on high-importance and post-disaster buildings by modifying the ductility-related force modification factor, R_d, even though such buildings are detailed to have a ductility capacity corresponding to the unmodified R_d factor. The reduced ductility demand means that such buildings will undergo less inelastic deformation than Normal Category buildings when subjected to the same level of earthquake ground motion. As a result, their SFRSs will sustain less damage and be more likely to remain functional after an earthquake. These qualities are important in buildings that are designated to be used to provide shelter or essential services following a disaster.

109. Structures designed to house essential services should remain operational immediately after an earthquake. However, the mere application of $I_E = 1.5$ for post-disaster buildings will not necessarily ensure the operational readiness of a facility after an earthquake. To determine what would be required for functional survival of a facility would entail a detailed study of what equipment and services need to be operational immediately after an earthquake and the anticipated behaviour of equipment and structural components during strong ground shaking. The study should address issues such as what equipment should be on emergency power, how long the emergency generators need to be able to run, how secure the fuel supply is, whether or not a stored supply of potable water is required. Building contents, such as equipment and services, that are required to remain functional immediately after an earthquake should be capable of accommodating the building deflections specified in NBC Article 4.1.8.18. (see also the Commentary section on NBC Article 4.1.8.18. starting at Paragraph 229).

110. The factor $I_E = 1.5$ for post-disaster buildings is not intended to cover the design considerations associated with special purpose structures, such as facilities for the manufacture or storage of toxic materials, whose failure could endanger the lives of a large number of people or affect the environment well beyond the confines of the building. These types of structures may require more sophisticated analysis.

111. Because earthquake loads are considered rare events (see the definition of earthquake load, E, in NBC Sentence 4.1.2.1.(1)), there is no general requirement for design at the serviceability limit states (SLS) level, and no SLS importance factors are given in NBC Table 4.1.8.5. However, post-disaster buildings must retain their capability to function following a major earthquake. So, rather than requiring SLS design for such buildings, their capability to continue to function is enhanced by specifying a reduced interstorey lateral deflection limit that is only 40% of that specified for Normal Category buildings (as discussed in the Commentary section on NBC Sentence 4.1.8.13.(3) starting at Paragraph 196).

Structural Configuration (NBC Article 4.1.8.6.)

112. The primary issue related to structural configuration is whether or not a structure is regular or irregular. Observations of earthquake damage to buildings indicate that, all other considerations being more or less equal, structures having regular SFRSs perform considerably better than those with irregularities. These observations are true even for structures that are well designed and built using good construction practices. The stiffness and mass irregularities also affect the dynamic behaviour of the structure. A dynamic analysis would usually provide a more realistic distribution of earthquake forces for structures with mass or stiffness irregularities than a static analysis, since the static analysis approach is based on regular structures.

113. There are several reasons why irregular structures behave poorly when subjected to strong earthquake ground motions. In a regular structure, strong ground shaking causes fairly predictable inelastic behaviour of the SFRS. However, in irregular structures, inelastic behaviour often tends to be less predictable and can be concentrated in the zones of irregularity, resulting in the structural elements in those zones being subjected to excessive deformation and, consequently, rapid failure. This effect is compounded by the fact that designers frequently overlook the potential stress concentrations in the zones of irregularity when detailing the structural system. Another reason why irregular structures behave poorly is that the elastic analysis normally used to distribute the demands of the earthquake ground motion throughout the structural system does not adequately predict the inelastic force and deformation demands in irregular structures, leading to inadequate design in the zones of irregularity. For these reasons, it is preferable that building designers use regular configurations and that gross irregularity be prohibited in locations of high seismicity where the expected very strong ground motions will put high inelastic demands on the structural system.

114. The NBC 2015 includes detailed definitions of nine types of irregularities and requirements for the design of buildings with such irregularities. In general, the presence of irregularities triggers restrictions and special requirements based on:

(1) the natural period or height of the building,
(2) the level of seismic hazard, i.e., the values of design spectral response acceleration, and/or
(3) the Importance Category of the building.

115. The restrictions and special requirements are of the following types:

(1) the particular type of irregularity is prohibited,
(2) design forces must be increased,
(3) the design must be based on dynamic analysis,
(4) special capacity design procedures are required for certain elements, and
(5) a special study involving non-linear analysis of the building is required.

NBC Sentence 4.1.8.6.(1)

116. The types of structural irregularities are detailed in NBC Table 4.1.8.6., the right-hand column of which references table notes that point to specific provisions, which state the applicable restrictions and special requirements. This approach is intended to assist the designer in consulting the applicable requirements for a particular type of irregularity, rather than needlessly examining the requirements for all types of irregularities.

117. The types of irregularities given in NBC Table 4.1.8.6. can be divided into two broad categories, namely vertical (elevation) and horizontal (plan) irregularities. Types 1 to 6 are vertical irregularities, and Types 7 and 8 are horizontal irregularities, while Type 9 can be either a horizontal or vertical irregularity. In addition, it should be noted that some structural configurations may result in two or more types of irregularities. For example, a building frame in which the upper storeys comprise a tower that is asymmetric in relation to the lower storeys would have both vertical geometric irregularity (Type 3) and torsional sensitivity (Type 7). These irregularities apply to all above-grade structures. Below-grade structures that include slabs surrounded by and attached to perimeter foundation walls which are in turn surrounded by soil tend to act as a stiff base attached to the ground and typically do not need to be considered in the definition of irregularity. An example of an exception to this principle is a structure located on a sloping site where portions around the perimeter of the stiff lower structure are open. In such a case, it is up to the designer to decide if it is necessary to consider the lower levels of the structure in the definition of irregularity. It is always important that sound engineering judgment be applied when assessing irregularity and its influence on structural response.

118. The vertical irregularities, Types 1 to 6, are described below:

Type 1 – Vertical Stiffness Irregularity: This type of irregularity exists when the lateral stiffness in any storey of an SFRS is less than 70% of the stiffness of any adjacent storey or less than 80% of the average stiffness of the three storeys directly above or below. Since ratios of the storey stiffnesses are considered, uncracked stiffnesses for concrete components may be used in qualifying an irregularity as Type 1. Note that an SFRS that is unchanged in dimensions over its full height is considered regular. For the purposes of this section, the lateral stiffness at a storey consists of both shear stiffness and flexural stiffness, which are considered separately; the structure is considered irregular if one or both do not meet the above criteria.

The stiffness of concrete and masonry shear walls can be determined as follows:

- For uncoupled walls, the flexural stiffness at a storey can be considered to be proportional to the sum of the moments of inertia multiplied by the modulus of elasticity of the walls at that storey for the direction considered.
- For coupled walls, the flexural stiffness at a storey is a function of the moments of inertia of each individual wall, the moment of inertia of the coupled system calculated using the area of the walls and the distance from the centroid of the coupled walls, and the modulus of elasticity of the walls at that storey for the direction considered. For coupled wall systems where the overall dimensions (except for thickness) do not change, the change in stiffness can be considered to be proportional to the change in wall thickness and modulus of elasticity of the walls at that storey for the direction considered. Partially coupled walls may be treated as frames.
- The shear stiffness at a storey can be considered to be proportional to the web area of the walls multiplied by the shear modulus at that storey for the direction considered.

The stiffness of wood-based shear walls can be determined using the approach given in CSA O86.

The flexural and shear stiffness of braced frames (including light-gauge steel braced frames) can be determined as follows:

- The flexural stiffness at a storey can be considered to be proportional to the modulus of elasticity multiplied by the sum of the moments of inertia of the frames, where the moment of inertia of each frame for the direction considered is calculated using the length of each frame and the area of each end column at that storey.
- The shear stiffness at a storey can be considered to be inversely proportional to the floor drift ratio calculated by applying a unit load, for the direction considered, to the isolated storey-height braces at that storey and restraining both vertical deformation of the columns and rotation of the floor about a vertical axis.

A general approach that can be used for braced frames, moment-resisting frames and plate walls consists of separating the shear and flexural stiffness calculations for each storey as follows:

- The shear stiffness can be determined as follows:
  (1) construct a computer model of the structure;
  (2) restrain the vertical deformation of the columns at each floor and roof and restrain the floors and roofs from rotating about the vertical axis;
  (3) apply a lateral load at the top of the structure in the direction of interest; and
  (4) use the floor drift ratio—the differential horizontal displacement between floors divided by the floor height—as a measure of the shear stiffness at each storey.
- The flexural stiffness at each storey can be determined as the sum of the moments of inertia of the frames multiplied by the modulus of elasticity, where the moment of inertia of each frame for the direction considered is calculated using the length of each frame and the area of each end column at that storey.

Boundary conditions at the base can affect the determination of regularity, particularly for moment-resisting frames if the columns are assumed to be fixed. If the frame is uniform for the first few storeys above the base, then the frame may be taken to be regular at the base. If the frame is not uniform for the first few storeys, then it should be defined as irregular or the assessment approach should be refined by considering more realistic boundary conditions at the column bases.

Type 2 – Weight (mass) Irregularity: This type of irregularity exists when the weight of any storey is more than 150% of the weight of an adjacent storey, with the exception that a roof with significantly less mass than the floor below is not considered to be irregular. A thicker floor slab supporting an intermediate mechanical floor is an example of a Type 2 irregularity. Another example is the transition from a lighter superstructure with a residential occupancy to a more massive parking garage below.

Type 3 – Vertical Geometric Irregularity: This type of irregularity exists when the horizontal dimension of the SFRS (not necessarily that of the building envelope) in any storey is more than 130% of its horizontal dimension in an adjacent storey. An example of a Type 3 irregularity is a reduction in the overall dimensions of a central elevator/stairwell core assembly that contains the SFRS of the building above a particular floor level. The stepping down of the horizontal dimensions of a moment-resisting frame below a certain floor level is another example. In many cases, such as in these examples, both Type 1 and Type 3 irregularities exist in the same SFRS.

Type 4 – In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element: Except for braced frames and moment-resisting frames, this type of irregularity exists when a lateral-force-resisting element in the SFRS has an in-plane offset or has a lower lateral stiffness than the element above it. While shears can be distributed to an offset SFRS using drag struts, the designer must be careful to provide a continuous load path for the overturning forces from the storeys above the offset. An example of an in-plane offset in an SFRS is where a system of walls descends between one set of column lines and at some level changes over to another set of column lines that is parallel to but offset from the first set. An example of a decrease in lateral stiffness in an SFRS below a particular level is where a system of walls terminates at that level of the building. Another example is a two-storey wall that has openings in one storey and either no openings or smaller ones in the adjacent storey.

Type 5 – Out-of-Plane Offsets: This type of irregularity exists when there is a discontinuity in the lateral force path, which is expected to remain in the plane of loading; an out-of-plane offset of the vertical elements of an SFRS produces such a discontinuity. An example of a Type 5 irregularity is a building that has a different spacing of column lines in its superstructure moment-resisting frame than in the frame of the parking garage below it. The relocation of the bracing in a steel frame from an exterior bay in lower storeys to an interior bay in upper storeys is another example. This type of irregularity is particularly problematic because of the large shear forces that must be transferred through the floor diaphragm at the level of the discontinuity; these cannot be calculated using a two-dimensional analysis, and even a three-dimensional elastic analysis (static or dynamic) cannot accurately estimate the magnitude of such large shear forces. While shears can be distributed to an offset SFRS using drag struts, the designer must be careful to provide a continuous load path for the overturning forces from the storeys above the offset.

Type 6 – Discontinuity in Capacity – Weak Storey: This type of irregularity exists when the storey shear strength in a storey is less than that in the storey above, thus allowing the formation of a sway mechanism in the weak storey. This is different than a soft storey, which is a stiffness issue and falls into a Type 1 and/or Type 3 irregularity. Note that, in keeping with the requirements of NBC Clause 4.1.8.3.(7)(c), elements in the storey above the weak storey that are not part of the SFRS but that have an adverse effect on its design must be accounted for. There have been many incidents of collapse due to the formation of a weak storey sway mechanism, which concentrates the displacement, non-linear demand and damage in the weak storey. An example of a Type 6 irregularity is a storey with overstrength walls above a weaker storey with moment-resisting frames or braced frames. Type 6 irregularities in moment-resisting frames and braced frames can be avoided by taking steps to ensure that the shear strength and shear demand are well matched throughout the building and that the shear sway strength in any storey is not less than that in the storey above it. As required by CSA S16, “Design of Steel Structures,” and CSA A23.3, strong continuous columns also help prevent a weak storey sway mechanism in moment-resisting and braced frames by forcing non-linear yielding behaviour to occur over several storeys. For flexural shear walls, a shear sway mechanism is prevented by providing sufficient shear strength to force a flexural hinge using capacity design principles. Walls yielding in flexure and forming a flexural hinge are not considered to be a sway mechanism and are therefore not a Type 6 irregularity. NBC Sentence 4.1.8.10.(1) prohibits this type of irregularity, except in locations of low seismicity; even then, the design forces must be increased significantly to accommodate the expected concentration of demand.

119. The common feature of vertical irregularities is that they result in non-uniform vertical distributions of stiffness, strength and/or mass, which, except for Type 5 irregularities, normally occur in the plane in which the design loads are applied. In these situations, the primary consequence for seismic design is that the distributions of the seismic forces and the resulting deformations throughout the height of the building are likely to be significantly different than those determined from the ESFP, which is based on the assumption that the stiffness, strength and mass are approximately uniform along the height of the building. Dynamic elastic analysis is usually required to obtain a suitable vertical distribution of seismic forces (see NBC Article 4.1.8.7. for exceptions). In addition, there are certain other restrictions imposed on structures with vertical irregularities, particularly post-disaster buildings; these are detailed in NBC Article 4.1.8.10. Extreme irregularities will result in high concentrations of non-linear deformations during strong ground shaking; such concentrations of deformations are not captured by elastic analysis.

120. The two types of horizontal irregularities, Types 7 and 8, and Type 9 irregularity are described below:

Type 7 – Torsional Sensitivity: This type of irregularity exists where structures with rigid diaphragms are torsionally flexible, which leads to large torsionally induced displacements.

The procedure for evaluating whether torsional sensitivity exists is described in NBC Sentence 4.1.8.11.(10). The ESFP does not adequately take into account the potential for large displacements in structures with torsional sensitivity, so dynamic analysis is usually required, as indicated in NBC Clause 4.1.8.11.(11)(b). NBC Article 4.1.8.7. describes the exceptions for which the ESFP is permissible.

Type 8 – Non-orthogonal Systems: This type of irregularity exists when the SFRS is not oriented along a set of orthogonal axes, which is the normal assumption with reference to the loads being considered to act independently along the two principal axes of the structure. The requirements for directions of loading that apply to this type of irregularity are given in NBC Sentence 4.1.8.8.(1).

Type 9 – Gravity-Induced Lateral Demand Irregularity: This type of irregularity exists in buildings with gravity-induced lateral demand (GILD) on the SFRS. Such buildings are more likely to experience severe damage during strong ground shaking because of their tendency to drift in the direction of the GILD, which leads to large residual displacements or instability. GILD can be imposed by a variety of gravity systems, including inclined or offset columns, cantilevered floor plates, and eccentric floor spans, each of which imposes very different gravity loads on different sides of the SFRS.

For the special case of inclined columns, a lateral displacement due to GILD induces a corresponding vertical displacement. The magnitude of the vertical displacement is a function of the column inclination; the larger the inclination from vertical, the larger the vertical displacement. For example, a column inclined at an angle of 45° would be expected to be displaced vertically by the same amount that it is displaced laterally during an earthquake. This vertical displacement will be imposed on all of the floors that are supported by the inclined column. The effects of the vertical displacement on the supported floors, including the vertical forces that are generated, must be considered in addition to the effects of GILD.

The susceptibility of a building to the amplification of displacements due to GILD, i.e., ratcheting behaviour, in one direction is related to the ratio α:

$$\alpha = \mathrm{Q_G}/\mathrm{Q_y}$$

where

$\mathrm{Q_G}$ = GILD on the SFRS at the critical level of the yielding system, and

$\mathrm{Q_y}$ = resistance of the yielding mechanism required to resist the minimum earthquake loads.

The force component, Q, selected to determine α will depend on the yielding mechanism of the SFRS. For example, for a wall system where the capacity of the building is limited by the overturning moment resistance, Q should be taken as the overturning moment, whereas for a steel-braced frame or a moment-resisting frame, Q should be taken as the storey shear at the critical level where column hinging or brace yielding is expected.

In the ratio α, the denominator, $\mathrm{Q_y}$, corresponds to the strength of the SFRS needed to resist earthquake forces alone (not including any extra capacity of the SFRS to resist GILD). $\mathrm{Q_y}$ can be determined by two different methods:

(1) from the lateral earthquake design force, V, multiplied by $\mathrm{R_o}$, or

(2) from the probable strength of the SFRS at the critical level of the yielding system minus the GILD on the SFRS at that level.

The first method provides a conservative value of α and is appropriate for determining whether a Type 9 irregularity is expected, prior to the detailed design of the SFRS.

Studies by Dupuis et al.[59] have demonstrated that the ratcheting behaviour associated with a Type 9 irregularity is dependent on the hysteretic response of the SFRS. Although SFRSs displaying significant energy dissipation through full hysteretic loops are generally considered to be favourable for seismic design, such systems are much more prone to ratcheting behaviour when subjected to GILD than systems that have self-centering characteristics demonstrated by flag-shaped or non-linear elastic hysteretic loops. As such, the definition of Type 9 irregularity in NBC Table 4.1.8.6. imposes different limits on α for SFRSs with self-centering characteristics and for other SFRSs. The SFRS should not be considered to have self-centering characteristics unless it can be demonstrated that there will be negligible residual strains after the expected level of cyclic damage. It should be noted that many state-of-the-art non-linear analysis programs do not correctly account for the increased residual strains due to damage, which prevent self-centering response.

Moment-resisting frames and coupled shear wall systems do not exhibit self-centering characteristics. Concrete cantilever (flexural) walls where a large fraction of the overturning resistance is provided by axial compression due to dead load, rather than by yielding of the vertical reinforcement, will have self-centering characteristics until the damage in the plastic hinge region becomes severe. Uncoupled concrete and masonry shear walls where a large fraction of the overturning resistance is provided by axial loads, rather than by yielding of the longitudinal reinforcement, can be considered as having self-centering characteristics.

Table J-2 summarizes how the NBC addresses SFRSs with a Type 9 irregularity. The amplification of displacements due to GILD is only taken into account for buildings with a Type 9 irregularity that are constructed in regions of high seismicity (i.e., where $I_EF_aS_a(0.2) \geq 0.5$), by multiplying deflections by 1.2. For such buildings with large values of α ($\alpha > 0.2$ for SFRSs with self-centering characteristics and $\alpha > 0.06$ for other SFRSs), the Linear Dynamic Analysis procedures of the NBC 2015 do not provide a reliable estimate of the displacement demands; however, non-linear time-step dynamic analyses performed according to NBC Article 4.1.8.12. can provide a more reliable estimate if the GILD is directly included in the model and special care is taken to adequately model the hysteretic characteristics of the SFRS. Designers are advised to use experimental data to assess the adequacy of the modeling of the hysteretic characteristics, which depend on many factors, including axial loads and section details.

Table J-2
Summary of Code Requirements for Gravity-Induced Lateral Demand Irregularity (Type 9) According to α Ratio

SFRSs with Self-centering Characteristics	Other SFRSs	Code Requirement
$\alpha \leq 0.1$	$\alpha \leq 0.03$	Not considered Type 9
$0.1 < \alpha \leq 0.2$	$0.03 < \alpha \leq 0.06$	Post-disaster buildings where $I_EF_aS_a(0.2) \geq 0.35$: Type 9 not allowed(1) Other buildings where $I_EF_aS_a(0.2) \geq 0.5$: multiply deflections by 1.2(2)
$\alpha > 0.2$	$\alpha > 0.06$	Post-disaster buildings where $I_EF_aS_a(0.2) \geq 0.35$: Type 9 not allowed(1) Other buildings where $I_EF_aS_a(0.2) \geq 0.5$: non-linear dynamic analysis studies required(3)

(1) See NBC Clause 4.1.8.10.(2)(a).
(2) See NBC Sentence 4.1.8.10.(6).
(3) See NBC Sentence 4.1.8.10.(7).

NBC Sentence 4.1.8.6.(2)

121. If none of the various types of irregularity described in NBC Table 4.1.8.6. occur in a structure, then it is classified as regular, which implies that the ESFP may be used for analysis, except in the case of tall buildings (height ≥ 60 m) with long fundamental lateral periods ($T_a \geq 2$ s) for which higher modes dominate response, as specified in NBC Clause 4.1.8.7.(1)(b).

NBC Sentence 4.1.8.6.(3)

122. Except as required by NBC Article 4.1.8.10., in situations where $I_EF_aS_a(0.2) < 0.35$, structures having any of the types of irregularities described in NBC Table 4.1.8.6. need not satisfy the Code provisions referenced therein. Because a value of $I_EF_aS_a(0.2) < 0.35$ indicates that the anticipated earthquake ground motions are relatively small, the restrictions specified for irregular structures are deemed to be unnecessary. For example, the use of dynamic analysis is not required for irregular structures where $I_EF_aS_a(0.2) < 0.35$ because the approximations inherent in the ESFP are unlikely to have serious consequences in situations where the ground motions are relatively small.

Signals for Special Requirements

123. NBC Sentence 4.1.8.6.(3) uses the product $I_EF_aS_a(0.2)$ ($F_a = F(0.2)$, as specified in NBC Sentence 4.1.8.4.(7)) as a signal for specific design and/or analysis requirements. The most common value for this short-period signal is 0.35, although values of 0.2 (NBC Sentence 4.1.8.10.(1)), 0.5 (NBC

Sentences 4.1.8.10.(6) and 4.1.8.10.(7)) and 0.75 (NBC Sentence 4.1.8.16.(8)) are also used. The NBC 2015 also uses the product $I_EF_vS_a(1.0)$ (F_v = F(1.0), as specified in NBC Sentence 4.1.8.4.(7)) as a signal with a value of 0.25 (NBC Sentence 4.1.8.10.(3)). In NBC Table 4.1.8.9., values of these signals delineate restrictions on the use of various types of SFRSs and limits on their heights for different ranges of seismicity.

124. To illustrate situations in which different requirements would be signalled, Table J-3 lists $I_EF_aS_a(0.2)$ and $I_EF_vS_a(1.0)$ for different combinations of location (Vancouver, Montréal or Toronto), Site Class (A, C or E) and earthquake importance factor, I_E (1.0 or 1.5). All combinations of I_E and Site Class in Montréal and Vancouver have values of $I_EF_aS_a(0.2)$ above 0.35, the most common short-period signal value. In Toronto, this short-period signal value is exceeded only for post-disaster buildings (I_E = 1.5) on Site Class C and for any building on Site Class E. The long-period signal value of $I_EF_vS_a(1.0)$ = 0.25 is not exceeded in Toronto; in Montréal, it is exceeded only for Site Class E, and in Vancouver, it is exceeded for all combinations of I_E and Site Class.

Table J-3
$I_EF_aS_a(0.2)$ and $I_EF_vS_a(1.0)$ for Vancouver, Montréal and Toronto According to Site Class and Earthquake Importance Factor

Site Class	I_E	$I_EF_aS_a(0.2)$			$I_EF_vS_a(1.0)$		
		Vancouver	Montréal	Toronto	Vancouver	Montréal	Toronto
A	1.0	0.64	0.44	0.17	0.30	0.06	0.026
	1.5	0.97	0.66	0.25	0.45	0.09	0.039
C	1.0	0.85	0.60	0.25	0.43	0.15	0.063
	1.5	1.27	0.89	0.37	0.65	0.23	0.095
E	1.0	0.82	0.62	0.38	0.68	0.26	0.16
	1.5	1.23	0.93	0.57	1.02	0.39	0.24

Methods of Analysis (NBC Article 4.1.8.7.)

NBC Sentence 4.1.8.7.(1)

125. As in the 2005 and 2010 editions of the NBC, the Dynamic Analysis Procedure is the default method of analysis in the NBC 2015, and the ESFP is permitted only if any of several specified criteria are met. The rationale for favouring the Dynamic Analysis Procedure is that structures respond to earthquakes dynamically rather than statically; overall response parameters (e.g., maximum lateral earthquake design force) and their distribution within the structure are affected by the structure's dynamic properties and the input ground motion. By contrast, the ESFP is only an approximate static simulation of this dynamic response and is reasonably accurate only in certain well-defined circumstances. For example, if the structure is uniform along its height and has a relatively short fundamental lateral period, T_a, then the static approximations for the natural period and for the height-wise distribution of forces within the structure are quite realistic. The ESFP may be used for analysis if any of the following criteria are met:

Clause (a): In cases where $I_EF_aS_a(0.2) < 0.35$, the ESFP may be used. In such cases, the approximations inherent in the ESFP are unlikely to have serious consequences for the relatively small ground motions. Even if the distribution of internal forces in long-period structures (with $T_a \geq 2$ s) determined using the ESFP is incorrect, in most instances, the resulting design will be satisfactory in regions of low seismicity.

Clause (b): Structures classified as regular (see the Commentary section on NBC Sentence 4.1.8.6.(2) in Paragraph 121) that are less than 60 m in height and that have a fundamental lateral period of less than 2 s may be analyzed using the ESFP. As noted previously, in most circumstances, regular structures are inherently suited to static analysis. The criteria in this Clause relating to height and fundamental lateral period reflect the fact that tall long-period structures respond to earthquake ground motions in the second or higher dynamic modes, rather than in the fundamental mode as assumed in the ESFP. Consequently, dynamic analysis is required for such structures, even if their configuration is regular.

Clause (c): Except for those with a Type 7 or 9 irregularity, structures classified as irregular (see the Commentary section on NBC Sentence 4.1.8.6.(1) starting at Paragraph 116) that are less than 20 m in height and that have a fundamental lateral period of less than 0.5 s may be analyzed using the ESFP. Static analysis is permissible in these cases both because irregularities have a minimal effect on the dynamic response of short-period structures and because the ESFP specified in NBC Article 4.1.8.11. is inherently somewhat conservative. The exclusion of structures with torsional sensitivity (Type 7 irregularity) reflects the fact that large displacements can occur in torsionally flexible structural systems, regardless of the fundamental lateral period of the structure.

Direction of Loading (NBC Article 4.1.8.8.)

NBC Sentence 4.1.8.8.(1)

126. Earthquake ground motions can originate from a source located in any horizontal direction from the site of a building. Consequently, for the purpose of designing structural elements so that they perform adequately when subjected to such ground motions, the loading on a building can be considered to act in any horizontal direction. However, ground motions often exhibit directionality (e.g., different amplitudes and frequency contents parallel and orthogonal to the direction of fault rupture). As such, the directions of loading of a structure should generally be selected to produce the most unfavourable effect on any structural element. In the NBC 2015, as in the NBC 2005 and NBC 2010, it is assumed that, for most building configurations, applying the specified loads independently along two orthogonal horizontal directions is sufficient for this purpose; NBC Clause 4.1.8.8.(1)(c) states the requirements for situations where this assumption is not applicable.

Clause (a): Where the components of the SFRS are oriented along a set of orthogonal axes, independent analyses must be performed about the two principal axes of the structure. As illustrated by DeVall,[10] the choice of axes can have a significant effect on the forces and moments in members of the SFRS; choosing an arbitrary set of orthogonal axes may result in member forces or moments that are significantly lower than those obtained using the principal axes.

Clause (b): Where the components of the SFRS are not oriented along a set of orthogonal axes, independent analyses may be performed about any two orthogonal axes, provided $I_E F_a S_a(0.2) < 0.35$. In situations where the ground motions are relatively small, the use of an arbitrary set of orthogonal axes is unlikely to have a significant effect on the ability of the resulting structure to perform adequately during an earthquake.

Clause (c): Where the components of the SFRS are not oriented along a set of orthogonal axes and $I_E F_a S_a(0.2) \geq 0.35$, the analysis procedure of NBC Clause 4.1.8.8.(1)(b) is not permitted because independent analyses about two arbitrary orthogonal axes may result in unconservative member forces or moments. In the procedure required by NBC Clause 4.1.8.8.(1)(c), the effects, Effect_x and Effect_y, (e.g., member forces or moments) due to the application of the specified earthquake loads independently in any two orthogonal directions, x and y, are considered, and the design is based on the most severe of the following combinations of these effects (i.e., that resulting in the greatest element strength):

$\pm 1.00\ \text{Effect}_x + \pm 0.30\ \text{Effect}_y$, or
$\pm 0.30\ \text{Effect}_x + \pm 1.00\ \text{Effect}_y$

The effects due to earthquake loads must be combined with those due to other loads in accordance with NBC Sentence 4.1.3.2.(2).

127. For beams, girders, slabs and other horizontal elements that resist loads primarily in one direction, the effects due to the application of earthquake loads in the orthogonal direction are normally small, but these orthogonal effects may be significant for columns and other vertical elements that resist loads in both directions, as discussed in FEMA 369, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 2: Commentary."

128. Earthquake ground motions may also contain a substantial vertical component. The vertical amplitude of a ground motion is typically 60% to 75% of its horizontal amplitude, but there are records of ground motions in which the vertical amplitude was similar to or larger than the horizontal amplitude. The ratio between the horizontal and vertical components of a ground motion is a frequency-dependent function that depends on the site conditions (Siddiqqi and Atkinson,[60] and Ghofrani et al.[61]) because horizontal ground motions experience greater site amplification than do vertical ones (Lermo and Chavez-Garcia[62]). Because buildings are very stiff in the vertical direction, vertical building periods are very short and normally produce little or no amplification of vertical ground motions. Buildings are also quite strong in the vertical direction; there is little history of damage due to vertical accelerations. For these reasons, the NBC does not require that buildings be designed to resist vertical ground motions. However, cantilevered building components may be sensitive to vertical accelerations; the loading of horizontally cantilevered floors, balconies and beams is specified in NBC Article 4.1.8.18.

SFRS Force Reduction Factors, System Overstrength Factors, and General Restrictions (NBC Article 4.1.8.9.)

129. Since the 2005 edition, the NBC has included two force modification factors: the ductility-related force modification factor, R_d, and the overstrength-related force modification factor, R_o. Mitchell et al.[9] provide the rationale for the maximum value of R_d and discuss similar factors in other building codes, such as EN 1998,[63] "Eurocode 8: Design of Structures for Earthquake Resistance," and FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions."

130. As discussed by Mitchell et al.,[9] structures have traditionally been designed so that their members have factored resistances that are equal to or greater than the effects due to factored loads. As a result, many structures, particularly those possessing a capacity for ductile behaviour, can have a considerable reserve of strength, which is not explicitly considered in the design process. In the NBC 2015, the DGMs are determined at a probability of exceedance of 2% in 50 years, and it is expected that the actual capacity of structures will be more or less fully utilized during such rare events. Consequently, it is reasonable to include the reserve strength in the design, provided it can be shown to exist. The overstrength-related force modification factor, R_o, represents the dependable or minimum overstrength that arises from the application of the design and detailing provisions of the relevant CSA standard referenced in NBC Table 4.1.8.9.

131. Mitchell et al.[9] show detailed calculations of R_o for various SFRSs and describe the component factors contributing to R_o: size (restricted choice of sizes of members and elements, and rounding of sizes and dimensions), the difference between nominal and factored resistances, the ratio of actual yield strength to minimum specified yield strength, the effect of strain hardening, and the effect of mobilizing the full capacity of a structural system by the formation of a collapse mechanism.

132. In the NBC 2015, several additions and modifications have been made to NBC Table 4.1.8.9.:

- entries for moderately ductile coupled walls and moderately ductile partially coupled walls have been added to correlate with new design and detailing requirements added to CSA A23.3 for such systems;
- the height restrictions on moment-resisting frames of conventional construction have been relaxed to reflect the design and detailing requirements in CSA A23.3;
- an entry for SFRSs using two-way slabs without beams has been added; the relatively low ductility-related force modification factor and the stringent height restrictions reflect the poor performance of such systems in earthquakes (Mitchell et al.[64]); and
- entries for SFRSs using tilt-up construction with different levels of ductility have been added because new requirements for the design and detailing of such systems, which are based on a number of studies (Lemieux et al.,[65] Devine et al.,[66] Dew et al.,[67] and Adebar et al.[68]), have been added to CSA A23.3. The height limits for such systems are based on construction practice in the Vancouver area.

NBC Sentence 4.1.8.9.(1)

133. This Sentence specifies that the values of R_d and R_o to be used in design must conform to those given for various SFRSs in NBC Table 4.1.8.9., and that the restrictions presented in the Table and all the requirements of NBC Subsection 4.1.8. must also be observed. For each structural material (i.e., steel, reinforced concrete, timber or masonry), the different types of SFRSs correspond to systems described in the applicable CSA standard; the structure must be designed and detailed in accordance with the standard in order to qualify for the listed values of R_d and R_o.

134. The values of R_d given in NBC Table 4.1.8.9. reflect the continuity and ductility provided by a particular SFRS. A value of R_d equal to 1.0 indicates that the SFRS exhibits little or no ductility; values of 1.0 have been assigned to systems that are not listed in NBC Table 4.1.8.9. because their ductility capacity has not yet been demonstrated. Values of R_d above 1.0 reflect the increased capability of the SFRS to accommodate inelastic cyclic deformations.

135. NBC Table 4.1.8.9. includes restrictions for different ranges of importance-modified short- and long-period design spectral response acceleration, $I_EF_aS_a(0.2)$ and $I_EF_vS_a(1.0)$, respectively (see the Commentary section titled Signals for Special Requirements starting at Paragraph 123). As explained in Table Note (2), the restrictions are indicated by either "NP," meaning "not permitted," or by a number representing the maximum permitted above-grade building height, in m; the absence of restrictions is indicated by "NL," meaning "not limited."

136. There are a few restrictions on SFRSs in regions of low seismicity where $I_EF_aS_a(0.2) < 0.2$: for example, for SFRSs not listed in NBC Table 4.1.8.9., a height limit of 15 m is imposed to limit risk in the case of unusual, unproven structural systems. NBC Sentence 4.1.8.9.(5) indicates that an alternative approach may be used to verify the performance of such unusual systems. In general, restrictions increase for higher ranges of $I_EF_aS_a(0.2)$ (there are no restrictions for the most ductile systems). Mitchell et al.[9] discuss the reasons for the various restrictions.

137. In choosing the SFRS for a building, large dissimilarities in the stiffness and ductility characteristics of the SFRS in the two orthogonal horizontal directions should be avoided. For example, the use of a flexible, ductile moment-resisting frame in one direction and limited-ductility masonry shear walls in the orthogonal direction would be unsuitable because seismic displacements induced in the frame would be likely to cause failure in the weak directions of the relatively brittle shear walls. On the other hand, the use of ductile reinforced concrete shear walls in one direction and moderately ductile shear walls in the orthogonal direction would be acceptable.

NBC Sentence 4.1.8.9.(2)

138. The values of R_o and R_d for an SFRS are interdependent in the sense that the product R_dR_o is an integral property of the SFRS. Consequently, as specified in NBC Sentence 4.1.8.9.(2), the value of R_o associated with each value of R_d in NBC Table 4.1.8.9. must be used: it is not permitted to use a different value of R_o determined through independent analysis, for example.

NBC Sentence 4.1.8.9.(3)

139. A building may include different types of SFRSs that are combined to resist lateral loads in the same direction. A common example of such a combination is a dual structural system comprising a moment-resisting frame and a shear wall or braced frame. NBC Sentence 4.1.8.9.(3) requires that the lowest value of the product R_dR_o be used when combinations of different types of SFRSs are acting in the same direction in the same storey: for example, the combination of a ductile steel moment-resisting frame with $R_dR_o = 7.5$ and a moderately ductile steel concentric braced frame with $R_dR_o = 3.9$ would require the use of the lower value of $R_dR_o = 3.9$ for the entire SFRS. Similarly, the combination of a moderately ductile reinforced concrete moment-resisting frame with $R_dR_o = 3.5$ and a moderately ductile reinforced concrete shear wall with $R_dR_o = 2.8$ would require the use of the lower value of $R_dR_o = 2.8$ for the entire SFRS. The purpose of NBC Sentence 4.1.8.9.(3) is to ensure that the lateral earthquake design force, V, is based on the SFRS with the lower value of R_dR_o, which will result in a higher value of V. Thus, the response of the system will be governed by its most vulnerable part, i.e., the part with the inferior combination of ductility capacity and overstrength.

140. The seismic forces on the two types of SFRSs in a dual structural system must be proportioned in accordance with the relative stiffnesses of the SFRSs, using the principles of structural mechanics. For dual structural systems in which the component SFRSs have different values of the ductility-related force modification factor, R_d, it is important to ensure that the less ductile SFRS can sustain the displacements associated with the more ductile SFRS without loss of strength. Also, if there are structural elements that are common to both SFRSs, then the detailing of those elements must meet the requirements for the more ductile of the two systems.

141. Dual structural systems may be designed so that 100% of the seismic load is carried by the system having the higher value of R_dR_o. If this design approach is followed, the other system, which is now not considered to be part of the SFRS, must be designed to retain its own functionality, i.e., to support its gravity loads while undergoing earthquake-induced deformations, as specified by NBC Sentence 4.1.8.3.(5).

NBC Sentence 4.1.8.9.(4)

142. A building may also incorporate different types of SFRSs along its height. For example, a ductile steel or concrete moment-resisting frame with a high R_dR_o value may be used in the upper tower part of a building, and a limited-ductility wall or braced frame system with a lower R_dR_o value may be used in the lower podium part of the building.

143. The provisions of NBC Sentence 4.1.8.9.(4) relating to vertical variations of R_dR_o along the height of a building were introduced to provide a practical design approach for cases where maintaining a constant R_dR_o value would be impractical and unnecessary for achieving good structural behaviour in an earthquake. The following are examples of such cases:
- (a) a ductile SFRS supported on a non-ductile foundation, which must be designed to have factored shear and overturning resistances that are greater than the lateral load capacity of the supported SFRS, in accordance with NBC Sentence 4.1.8.16.(2);
- (b) a ductile above-grade SFRS over a strong and stiff below-grade structure surrounded by walls;
- (c) a tall ductile structure over a low above-grade podium, particularly a podium that contains additional walls and lateral elements; and
- (d) a ductile wood-frame shear wall structure over a stiff, limited-ductility one- or two-storey concrete structure.

144. It is not appropriate simply to take a load distribution determined by a linear static or dynamic analysis based on an R_dR_o value of 1.0, to divide the upper storey loads in the distribution by the larger R_dR_o value, and to divide the lower storey loads in the distribution by the smaller R_dR_o value. In general, a non-linear analysis is required. However, simple, approximate and conservative linear approaches for two special cases are outlined in the following:
- (1) For regular structures where the change in R_dR_o is near grade, analyze the entire structure using the ESFP or the Modal Response Spectrum Method for the forces calculated using both values of R_dR_o, design the upper part of the structure for the forces calculated using the larger R_dR_o value, and design the lower part of the structure for the larger of
  - (a) the forces from the entire structure calculated using the smaller R_dR_o value, and
  - (b) the forces related to the lateral capacity of the upper part of the structure.
- (2) For structures described in case (d) in Paragraph 143 for which the stiffness of the storey(s) in the lower structure is greater than three times that of each of the storeys in the upper structure:
  - (a) follow approach 1(a), but use the Modal Response Spectrum Method for analysis; or
  - (b) where permitted, use the ESFP; idealize the upper structure as a separate building with a fixed base starting at the top of the lower structure and with a period appropriate for its height; analyze this building for the forces calculated using the larger R_dR_o value; idealize the lower structure as a separate short building with a period appropriate for its height; analyze this short building for the forces calculated using the smaller R_dR_o value with the addition of the forces generated by applying the lateral capacity calculated at the base of the upper structure as a load to the top of the lower structure.

145. In both of these special cases, the design forces need not exceed those calculated using an R_dR_o value of 1.3, but a weak storey is not permitted.

For all structures with vertical variations of R_dR_o, the total height of the structure must not exceed the limit for the larger R_dR_o value, and the height of the lower portion of the structure must not exceed the limit for the smaller R_dR_o value.

NBC Sentence 4.1.8.9.(5)

146. Only the most common types of SFRSs are addressed in NBC Table 4.1.8.9. If an SFRS that is not specifically identified in the Table is used, then $R_d = R_o = 1.0$ must be used for design; this requirement is based on the assumption that systems that are not defined in the Table should be designed conservatively. If it can be demonstrated through testing, research and analysis that the performance of a structural system is at least equivalent to that of an SFRS listed in NBC Table 4.1.8.9., then NBC Sentence 4.1.8.9.(5) allows the R_d and R_o values for that SFRS to be used.

147. The most common approach for establishing the appropriate value of the ductility-related force modification factor, R_d, for a structural system is by cyclic testing of its elements and sub-assemblages, which involves subjecting them to a number of cycles of reversing deformations that increase until the capacity is reached. The evaluation of these test results and the analysis of typical building configurations incorporating the elements and sub-assemblages are then used to determine the expected seismic performance of a building's structural system, primarily the overall displacement ductility capacity. Examples of such an approach are given by Mitchell and Paultre[69] and by Rahgozar and Humar.[70] In accordance with NBC Sentence 4.1.8.9.(5), the seismic performance so determined must be at least equivalent to that of an SFRS listed in NBC Table 4.1.8.9. for the corresponding R_d value to be permitted to be used in design.

148. The overstrength-related force modification factor, R_o, can be determined using the methodology described by Mitchell et al.[9] Caution needs to be exercised to ensure that minimum or dependable values of the various component factors are used. Some of these component factors may be determined from further evaluation of the results of tests used in the process of determining R_d; in any case, the component factors should be based on assumptions that are compatible with the test results. The R_o value determined by the methodology of Mitchell et al.[9] must be comparable to that for the equivalent SFRS listed in NBC Table 4.1.8.9.

Additional System Restrictions (NBC Article 4.1.8.10.)

NBC Sentence 4.1.8.10.(1)

149. As noted in Paragraph 118, structures with a Type 6 irregularity (Discontinuity in Capacity – Weak Storey) as described in NBC Table 4.1.8.6. are particularly vulnerable to damage and collapse during seismic ground motions. NBC Sentence 4.1.8.10.(1) prohibits such structures except in regions of low seismicity where $I_EF_aS_a(0.2) < 0.2$. The forces used for the design of the SFRS for such structures, where permitted, must be multiplied by R_dR_o to ensure that the SFRS remains elastic when subjected to DGMs. NBC Clause 4.1.8.10.(2)(b) prohibits Type 6 irregularities in post-disaster buildings.

NBC Sentence 4.1.8.10.(2)

150. Special consideration is given to post-disaster buildings through the specification of an earthquake importance factor, I_E, of 1.5 in NBC Article 4.1.8.5. In addition, NBC Sentence 4.1.8.10.(2) imposes other restrictions on the design of the SFRS for such buildings. The intention of these restrictions is to increase the likelihood that such buildings will remain operational immediately after an earthquake by avoiding more vulnerable structural forms or types of structures.

Clause (a): This Clause prohibits most types of irregularities in post-disaster buildings in regions of moderate to high seismicity where $I_EF_aS_a(0.2) \geq 0.35$. The irregularities that are prohibited—Types 1, 3, 4, 5, 7 and 9 as described in NBC Table 4.1.8.6.—are those characterized by geometric or stiffness discontinuities, which can lead to localized concentrations of inelastic deformation, torsional sensitivity or GILD.

Clause (b): This Clause prohibits the Type 6 irregularity (Discontinuity in Capacity – Weak Storey) in post-disaster buildings.

Clause (c): This Clause requires that post-disaster buildings have an SFRS with $R_d \geq 2.0$. Such SFRSs have at least limited ductility, which gives them at least a minimal capability to dissipate energy through inelastic deformation and provides some protection against ground motions that exceed the design level.

Clause (d): This Clause prevents the construction of a soft storey in post-disaster buildings by requiring that the lateral stiffness of any supporting storey be not less than that of the storey above it.

NBC Sentence 4.1.8.10.(3)

151. This Sentence requires that, for buildings having a fundamental lateral period, T_a, of 1.0 s or greater, where $I_EF_vS_a(1.0) > 0.25$, shear walls that are not wood-based and that form part of the SFRS be continuous from their top to the foundation and not have an irregularity of Type 4 (In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element) or Type 5 (Out-of-Plane Offsets) as described in NBC Table 4.1.8.6. In an earthquake, the presence of discontinuous walls can lead to significant damage to supporting columns or transfer systems. The prohibition of these discontinuities is intended to ensure that shear walls in taller (long-period) buildings function effectively during strong earthquake shaking. The requirements of NBC Sentence 4.1.8.10.(3) also apply to cold-formed steel shear walls designed and detailed in accordance with CSA S136, "North American Specification for the Design of Cold-Formed Steel Structural Members," that have cold-formed steel studs and wood-based panels (see NBC Table 4.1.8.9.).

NBC Sentence 4.1.8.10.(4)

152. This Sentence prohibits Type 4 and 5 irregularities, as described in NBC Table 4.1.8.6., in the SFRS of buildings constructed with more than 4 storeys of continuous wood construction, where $I_EF_aS_a(0.2) \geq 0.35$. These types of irregularities are prohibited to ensure that the expected structural response is maintained at a reasonable level by a well-defined SFRS.

Determining the Number of Storeys for the Purpose of NBC Sentences 4.1.8.10.(4), 4.1.8.11.(12) and 4.1.8.12.(12)

153. NBC Sentences 4.1.8.10.(4), 4.1.8.11.(12) and 4.1.8.12.(12) apply to buildings constructed with more than 4 storeys of continuous wood construction up to a maximum of 6 storeys. For the purpose of determining the number of storeys for the application of these Sentences, all storeys of continuous wood construction above the base need to be considered, including any full or partial storeys of wood construction below the first storey, and wood cripple walls (sometimes referred to as knee walls or pony walls) are to be considered as a full storey. Many factors can affect the location of the base, including the slope and location of the grade, the location and stiffness of the SFRS elements, openings in basement walls, and the proximity to adjacent buildings. Additional information on determining the location of the base can be found in ASCE/SEI 7. Unless the subdivided portions of a building are separated in accordance with the requirements of NBC Article 4.1.8.14., a building subdivided with firewalls is considered as one building for the purpose of determining the number of storeys in the application of the above-noted Sentences. See Figure J-21 for examples of 5-storey buildings.

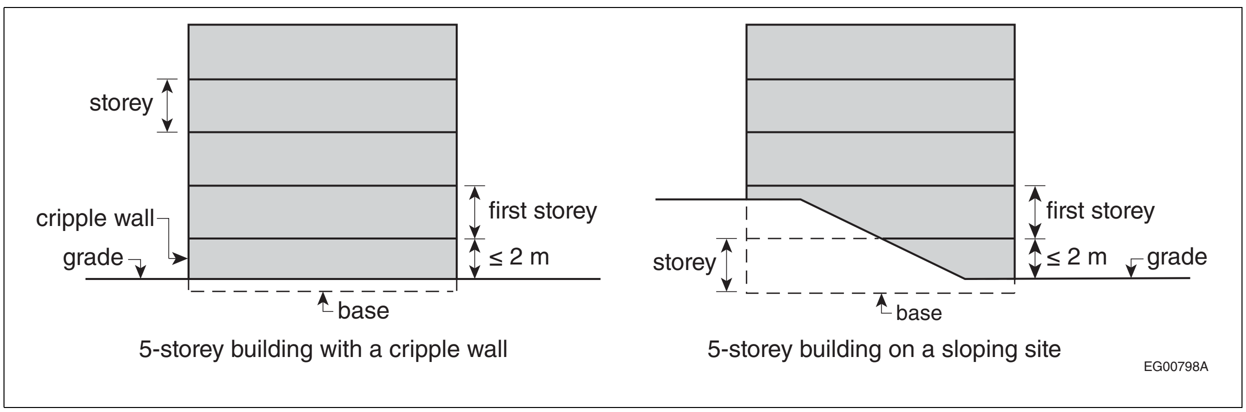


Figure J-21
Examples of 5-storey buildings for the purposes of NBC Sentences 4.1.8.10.(4), 4.1.8.11.(12) and 4.1.8.12.(12)

Cripple Walls

154. A cripple wall is a short stud wall between the foundation and the floor system above it. Although cripple walls can be made with different materials, they are commonly found in wood-frame structures. Figures J-22 and J-23 show a common configuration in which a concrete foundation wall extends from a footing to slightly above grade and supports a wood-frame cripple wall that extends up to the wood floor system. The design of such a wall system must account for instability in both the in-plane and out-of-plane horizontal directions to prevent a hinge from forming at the connection between the cripple wall and the foundation wall, and to ensure overall stability. Sufficient bracing or sheathing is required to resist in-plane lateral forces and to prevent the cripple wall from hinging and potentially collapsing (see Figure J-22).

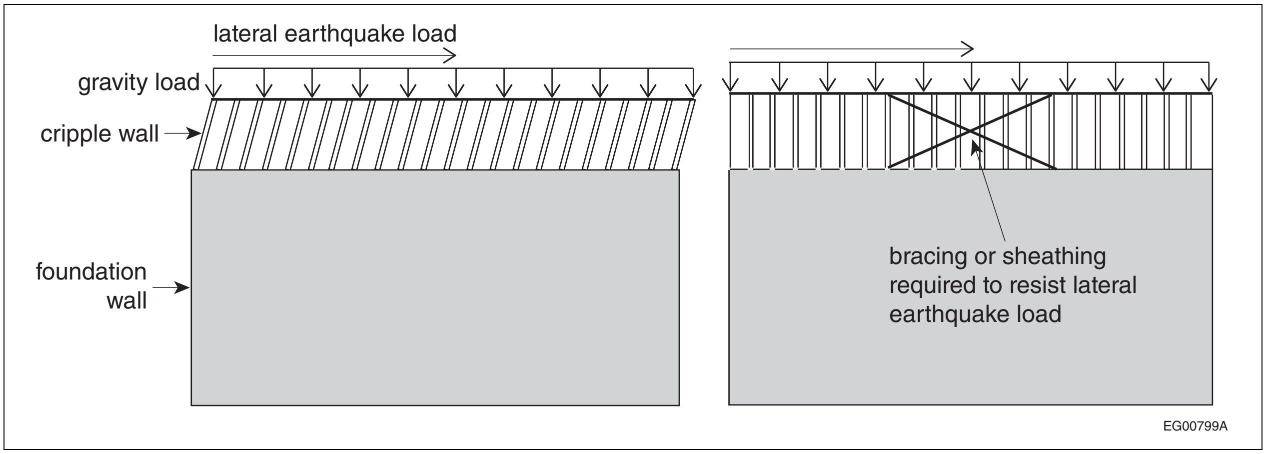


Figure J-22
Stabilizing a cripple wall against in-plane failure

Designing the foundation wall to cantilever from its base or to span horizontally between buttress or return walls will help the cripple wall resist out-of-plane lateral loads (see Figure J-23). Extending the concrete foundation wall up to the underside of the wood floor system will help ensure that the cripple wall is able to resist both in-plane and out-of-plane lateral forces. It should be noted that the load combinations given in NBC Table 4.1.3.2.-A need to be considered when designing such wall systems for in-plane and out-of-plane stability.

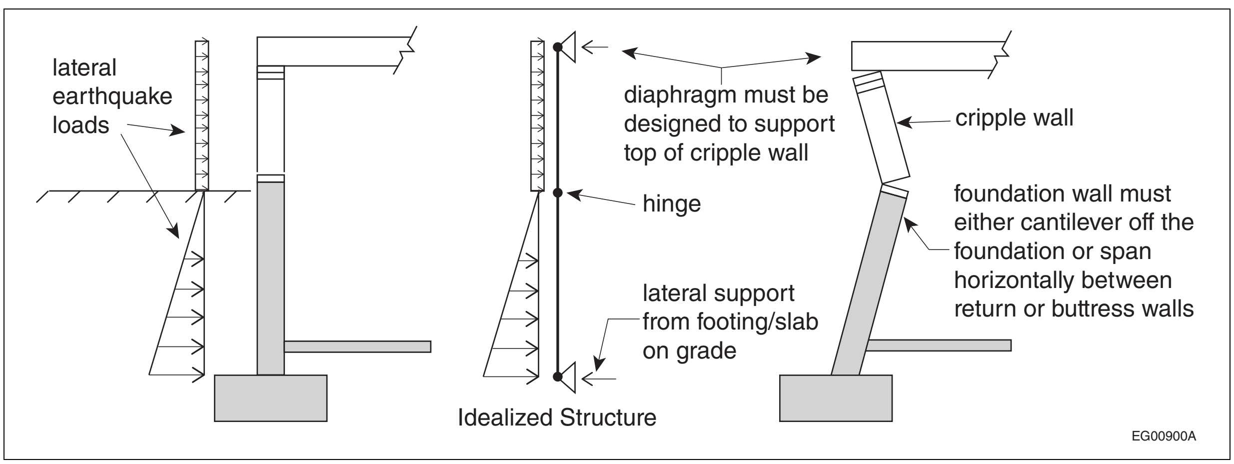


Figure J-23
Stabilizing a cripple wall against out-of-plane failure

Equivalent Static Force Procedure for Structures Satisfying the Conditions of NBC Article 4.1.8.7. (NBC Article 4.1.8.11.)

NBC Sentence 4.1.8.11.(1)

155. As described in the Commentary section on NBC Article 4.1.8.7. (see Paragraph 125), the ESFP can be used under certain conditions in lieu of dynamic analysis to determine the design earthquake actions (i.e., forces in elements and structural deformations). NBC Sentence 4.1.8.11.(1) specifies that the static earthquake loads must be determined in accordance with the procedures given in NBC Article 4.1.8.11. The lateral loads are to be applied to a linear mathematical model of the SFRS in the directions specified in NBC Article 4.1.8.8. The model must meet the requirements of NBC Sentence 4.1.8.3.(8) and must include appropriate modeling of the interface between the SFRS and the foundation. A detailed description of the ESFP specified in NBC Article 4.1.8.11. is given by Humar and Mahgoub.[71]

156. The static loading specified in NBC Article 4.1.8.11. is intended to approximate dynamic effects in a rational manner. However, such an approximation may not be valid in certain circumstances, in which case dynamic analysis is required, as specified in other Articles in Subsection 4.1.8. In particular, the ESFP assumes that the response of the structure is predominantly in the fundamental mode; the effects of the participation of higher modes are then incorporated by modifying the fundamental mode behaviour. If the response is not predominantly in the fundamental mode, as in the case of tall long-period structures, then the ESFP is not appropriate; dynamic analysis is required for such structures, as specified in NBC Clause 4.1.8.7.(1)(b).

NBC Sentence 4.1.8.11.(2)

157. For a structure with a fundamental lateral period, T_a, the minimum lateral earthquake design force, V—often referred to as the design base shear—is calculated in accordance with the following formula:

$$V = S(T_a)\,M_v I_E W/(R_d R_o)$$

where

W = weight of the structure,

$I_E S(T_a)$ = importance-modified design spectral response acceleration, which when multiplied by W, represents the maximum lateral earthquake force in an elastic single-degree-of-freedom system with a period T_a,

M_v = higher mode factor, which accounts for the participation of higher modes in the dynamic response of the structure, such that the product $S(T_a)M_vI_EW$ represents the maximum lateral earthquake force in an elastic multi-degree-of-freedom system with a period T_a, and

R_dR_o = reduction factor, which accounts for both ductility and overstrength, as discussed in the Commentary section on NBC Article 4.1.8.9. (starting at Paragraph 129) (the rationale for reducing the maximum force by placing this product in the denominator of the expression for V is given by Mitchell et al.[9]).

Clause (a): Because of the uncertainty associated with the determination of earthquake-induced forces and deflections in tall long-period buildings, this Clause specifies a minimum value of V that corresponds to its value at T = 4.0 s, even though at periods greater than 4.0 s, the design spectral response acceleration, S(T), decreases with each increase in period. This Clause does not apply to steel plate walls or wood-based shear walls for which the minimum lateral earthquake design force, V, is governed by NBC Clause 4.1.8.11.(2)(b).

Clause (b): For long-period structures with moment-resisting frames, braced frames or other SFRSs, such as steel plate walls or wood-based shear walls, there is concern that the ductility demand may not be uniformly distributed along the height of the SFRS and that the concentration of such demand in a single storey may lead to the development of a weak storey. To minimize the likelihood of the formation of a weak storey and premature collapse, this Clause specifies a minimum value of V for such systems that corresponds to its value at T = 2.0 s. Systems such as reinforced concrete shear walls (individual or coupled) are less prone to weak-storey response and may be designed in accordance with Clause 4.1.8.11.(2)(a).

Clause (c): This Clause specifies a cap on the minimum lateral earthquake design force for short-period buildings. Experience has demonstrated that damage to well-designed short-period structures, even those with limited ductility, is rare during earthquakes. When it does occur, damage in such structures results from deformations, rather than directly from high force levels, but the deformations in such structures are usually too small to cause damage because spectral response displacements at short periods are very small. Also, the actual excitation of such structures is likely to be less than that predicted by the specified spectral response acceleration, owing to factors such as finite foundation size and energy dissipation at the foundation–structure interface, e.g., due to sliding or radiation damping.

Short-period structures tend to have sources of both strength and deformability that are not readily quantified in a simplified analysis and that increase their ability to survive major earthquakes (NZS 4203:1992,[72] “General Structural Design and Design Loadings for Buildings”). Because such structures are inherently stiff, they do not typically undergo deformations that cause significant damage, particularly if there is some ductility capacity in the structural system. Very brittle short-period structures would, of course, not behave as well because cracking leading to failure can occur without significant deformation.

For these reasons, this Clause provides two formulae—one of which applies an experience-based reduction factor of 2/3 to the maximum short-period base shear—which are used to limit forces in all but the most brittle structural systems. For an SFRS having a value of $R_d \geq 1.5$, the minimum lateral earthquake design force need not exceed the larger of the following values:

$$(2/3)\,S(0.2)\,I_E W/(R_d R_o)\,,\text{ and}$$
$$S(0.5)\,I_E W/(R_d R_o)$$

The higher mode factor, M_v, is not included in these expressions because its value is 1.0 for $T_a \leq 0.5$ s (see NBC Table 4.1.8.11.). The second expression has been introduced in the NBC 2015 to ensure that the short-period cap of (2/3)S(0.2) is not extended to periods longer than 0.5 s for which the cap was not intended. In cases where the spectral shape at short periods is flat, the short-period cap would extend beyond T = 0.5 s if the first expression were used (see Figure J-24).

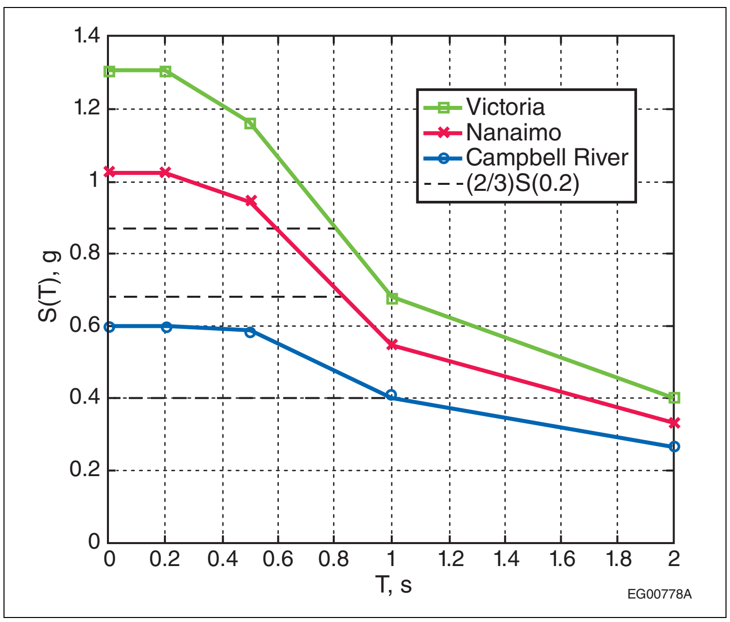


Figure J-24
Examples of design response spectra for Site Class C where the short-period cap of (2/3)S(0.2) would extend past T = 0.5 s

NBC Sentence 4.1.8.11.(3)

158. In the ESFP, the expression for the minimum lateral earthquake design force, V, includes the design spectral response acceleration, $S(T_a)$, determined at the fundamental lateral period, T_a, in the direction of loading. The approach used to determine T_a is particularly significant in the short- to medium-period range ($0.2 \text{ s} \leq T \leq 1.0$ s), in which spectral response acceleration, $S_a(T)$, declines steeply as a function of period, T. In most cases, it is permissible to calculate T_a by using empirical formulae based on building geometry (height, h_n, or number of storeys, N), which are specified in NBC Clauses 4.1.8.11.(3)(a) to (c) (Saatcioglu and Humar[12] discuss the rationale for these approximations). The value of T_a determined from the empirical formulae is to be used only for estimating seismic design loads and not for the determination of wind forces.

Clause (a): This Clause provides empirical formulae for moment-resisting frames, which remain unchanged from the NBC 2010, namely height-based formulae to be used for steel and concrete moment-resisting frames, and a storey-based formula to be used only for other moment-resisting frames.

Clause (b): This Clause provides a height-based empirical formula for braced frames, which was developed for the NBC 2005 because the formula specified in NBC Clause 4.1.8.11.(3)(c) was found to be unduly conservative for such systems.

Clause (c): This Clause provides a height-based empirical formula for shear walls and other structures, which remains unchanged from the NBC 2010.

Clause (d): As an alternative to the empirical formulae of NBC Clauses 4.1.8.11.(3)(a) to (c), established methods of mechanics may be used to determine T_a. However, such methods must use an appropriate structural model that meets the requirements of NBC Sentence 4.1.8.3.(8) (see the Commentary section on that Sentence starting at Paragraph 90). Even if these requirements are met, calculated fundamental lateral periods tend to be longer than those measured in actual structures because modeling usually does not take into account the participation of non-structural elements, which tend to stiffen the structure. The use of calculated periods that are longer than actual periods results in non-conservative seismic design forces because the design spectral response acceleration is lower at the longer periods, which leads to a lower minimum lateral earthquake design force. To guard against excessively long periods, NBC Clause 4.1.8.11.(3)(d) requires that the calculated fundamental lateral periods not exceed a certain magnitude of the value determined in NBC Clause 4.1.8.11.(3)(a), (b) or (c), depending on the type of SFRS. A larger limit is allowed for shear wall structures because studies have shown that periods calculated for such structures using established methods of mechanics are similar to measured values (Saatcioglu and Humar[12]).

Although the use of shorter fundamental lateral periods produces conservative seismic design forces, it produces non-conservative results with respect to the determination of deflections. As such, the use of unrealistically short periods may result in a significant underestimation of lateral deflections and interstorey drifts, which would be problematic for flexible structural systems in which deformations are likely to govern performance, e.g., moment-resisting frames.

The upper limits on the calculated fundamental lateral periods prescribed in NBC Clause 4.1.8.11.(3)(d) are used to account for the possibility that the actual structure may be stiffer than the model used to calculate the period and may therefore attract higher earthquake forces. The deflections calculated by applying these higher forces on the flexible model are quite conservative. For consistency, the model used to calculate the deflections should be the same as the one used to calculate the fundamental lateral period and, hence, the earthquake forces. Therefore, in calculating the deflections, the fundamental lateral period determined according to NBC Clause 4.1.8.11.(3)(d) may be used without the upper limit specified in NBC Subclauses 4.1.8.11.(3)(d)(i) to (d)(iv). However, as specified in NBC Sentence 4.1.8.11.(2), the minimum lateral earthquake design force for which the deflections are calculated must not be less than the force corresponding to $T_a = 4$ s for walls, coupled walls and wall-frame systems, and $T_a = 2$ s for moment-resisting frames, braced frames and other systems. Accordingly, NBC Subclause 4.1.8.11.(3)(d)(v) specifies an upper limit on the calculated fundamental lateral period of 4 s and 2 s, respectively, for such systems.

NBC Sentence 4.1.8.11.(4)

159. New formulae for determining the fundamental lateral period of single-storey buildings with flexible roof diaphragms of steel deck or wood have been introduced in the NBC 2015. Experimental and numerical studies have shown that such buildings have inherently higher lateral flexibility because of the in-plane deformations of the diaphragm under lateral loading. As such, they have longer fundamental lateral periods than buildings of the same height with rigid diaphragms (Tremblay and Stiemer,[73] Medhekar and Kennedy,[74] Tremblay et al.,[75][76] Tremblay and Rogers,[77] and Lamarche et al.[78]). The flexibility of the diaphragm generally increases with its span. In addition, because most of the seismic weight is concentrated at the roof level in such buildings, they have longer periods than buildings of the same height in which the seismic weight is distributed along the height of the building.

160. These effects can be accounted for through dynamic analysis using a structural model that explicitly includes the geometry and the shear and flexural properties of the roof diaphragm. Simplified approaches and expressions to account for these effects in calculating the fundamental lateral period of simple buildings have been proposed by Medhekar and Kennedy,[74] Lamarche et al.,[78] Humar and Popovski,[79] and Wilson et al.[80] According to the approach described in FEMA 356, “Prestandard and Commentary for the Seismic Rehabilitation of Buildings,” and Trudel-Languedoc et al.,[81] the fundamental lateral period, T_a, can be estimated as follows:

$$T_a = 2\pi\sqrt{\frac{W}{gV}\left(\Delta_B + 0.76\Delta_D\right)}$$

where
- W = weight of the building,
- g = gravitational acceleration,
- V = minimum lateral earthquake design force,
- Δ_B = average storey drift of the vertical SFRS elements adjoining the diaphragm, calculated with V uniformly distributed along the diaphragm span, L, and
- Δ_D = maximum in-plane deflection of the diaphragm relative to the adjoining vertical SFRS elements (see Figure J-25).

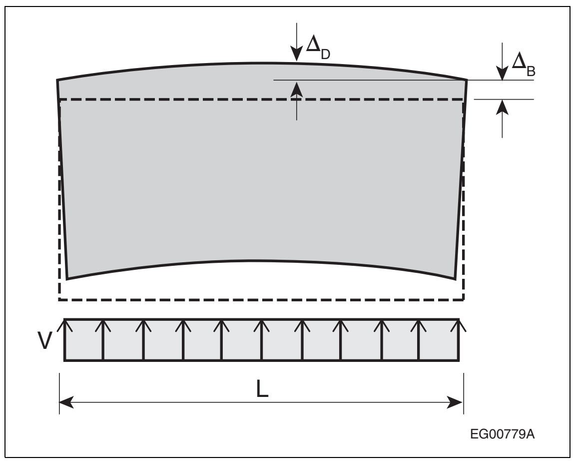


Figure J-25
Plan view of the in-plane deflection of a flexible roof diaphragm, Δ_D, and the storey drift of the adjoining vertical SFRS elements, Δ_B

161. According to NBC Sentence 4.1.8.11.(4), a lower-bound estimate of the fundamental lateral period of buildings with flexible roof diaphragms and with certain types of SFRSs may be obtained by adding the term 0.004L, where L is the shortest length of the diaphragm between adjacent vertical elements of the SFRS, to the fundamental lateral period determined for the SFRS.

Clause (a): This Clause provides an empirical formula for determining the fundamental lateral period of buildings with flexible roof diaphragms in which the SFRS consists of shear walls. The empirical formula is the same as the one specified in NBC Clause 4.1.8.11.(3)(c), except for the addition of the term 0.004L.

Clause (b): This Clause provides an empirical formula for determining the fundamental lateral period of buildings with flexible roof diaphragms in which the SFRS consists of steel moment-resisting frames or steel braced frames. The empirical formula is similar to the one specified in NBC Clause 4.1.8.11.(3)(b), except for the addition of the term 0.004L and an increase in the height term to provide better agreement with analytically and experimentally determined periods for such buildings.

Clause (c): The fundamental lateral period of single-storey buildings with flexible roof diaphragms may also be determined by established methods of mechanics. The value of T_a so determined must not be greater than 1.5 times the value obtained using the empirical formula of NBC Clause 4.1.8.11.(3)(a) or (b), as applicable, for reasons that are similar to those given in the Commentary section on NBC Clause 4.1.8.11.(3)(d) (see Paragraph 158).

The flexibility of the roof diaphragm may also lead to a magnified ductility demand on the vertical elements of the SFRS and to increased forces and deflections in the roof diaphragm. The additional requirements of NBC Sentence 4.1.8.15.(4) must be satisfied for single-storey buildings with flexible roof diaphragms of steel deck or wood that are designed with a value of R_d greater than 1.5, where the calculated maximum relative deflection, Δ_D, of the diaphragm under lateral loads exceeds 50% of the average storey drift, Δ_B, of the adjoining vertical elements of the SFRS (i.e., $\Delta_D/\Delta_B > 0.5$).

NBC Sentence 4.1.8.11.(5)

162. The weight of the building, W, used in the formula for the minimum lateral earthquake design force, V, is calculated as the sum of the weights of each of the storeys in the building, W_i (see NBC Article 4.1.8.2. for the definition of W). If the design of the building includes permanent masses that are normally included in the description of live load, then the weight of these masses should be inculded in the calculation of W.

NBC Sentence 4.1.8.11.(6)

163. As noted in the Commentary section on NBC Sentence 4.1.8.11.(2) (see Paragraph 157), the calculation of the minimum lateral earthquake design force involves the transformation of the force in a single-degree-of-freedom system to that in a multi-degree-of-freedom system by the inclusion of a higher mode factor, M_v, to account for the participation of higher modes in the dynamic response of the structure. The extent of the participation of higher modes is a function of the type of SFRS, the fundamental lateral period of the structure, and the shape of the design response spectrum, which is respresented by the spectral ratio, S(0.2)/S(5.0). NBC Table 4.1.8.11. specifies values of M_v for different combinations of these parameters. The methodology for calculating M_v described by Humar and Mahgoub[71] has been further refined for the NBC 2015, partly to account for wide variations now present in the spectral shape, such that the spectral ratio, S(0.2)/S(5.0), ranges from about 5 to 65 and now captures the effect of site coefficients on the spectral shape.

164. As shown by NBC Table 4.1.8.11., higher mode effects are most significant for long-period wall systems; the maximum value of M_v in the Table is 4.65 for walls and wall-frame systems where S(0.2)/S(5.0) = 65. The value of M_v is 1.0 for all structures with $T_a \leq 0.5$ s because higher mode effects are insignificant for such structures. In fact, Humar and Mahgoub[71] show that the calculated value of M_v for such short-period structures is typically 0.8 or less. The benefit of the reduced lateral earthquake design force resulting from a lower calculated value of M_v may be obtained by using dynamic analysis; for regular structures, NBC Sentence 4.1.8.12.(8) permits the dynamically determined lateral earthquake design force to be as low as 80% of the statically determined value.

165. In long-period structures, higher mode effects tend to increase the lateral earthquake design force, V, from that calculated for a single-degree-of-freedom system. The use of M_v to account for higher mode effects in the calculation of V leads to an overvaluing of the overturning moments in the structure. Although the contribution of higher modes to V can be significant, the corresponding contribution of higher modes to the base overturning moment is relatively small. Consequently, it is necessary to reduce the base overturning moment by applying a base overturning reduction factor, J (see NBC Table 4.1.8.11). Humar and Mahgoub[71] describe a methodology for determining J, which has been further refined for the NBC 2015.

166. NBC Table 4.1.8.11. specifies values of M_v and J for five types of SFRSs, as a function of fundamental lateral period, T_a, and spectral ratio, S(0.2)/S(5.0). Because values of M_v and J are only provided for certain values of T_a and S(0.2)/S(5.0), and because values of F(T) are only provided for certain values of T, several interpolations are necessary to determine $S(T_a)M_v$ and J for a building having an intermediate value of T_a, as illustrated by the following steps:

Step 1: Determine the fundamental lateral period, T_a, of the building, and obtain values of PGA, $S_a(0.2)$, $S_a(5.0)$, $S_a(T_1)$ and $S_a(T_2)$ for the location of the building from NBC Table C-3, where T_1 is the period in the Table that is closest to and lower than T_a, and T_2 is the period in the Table that is closest to and higher than T_a. For coupled walls, walls and wall-frame systems where $T_a > 4$ s, use $T_a = 4$ s, $T_1 = 2.0$ s and $T_2 = 5.0$ s. For moment-resisting frames, braced frames and other systems where $T_a > 2$ s, use $T_a = 2$ s (T_1 and T_2 are not required).

Step 2: Using the values of PGA and $S_a(0.2)$, determine PGA_{ref} as 0.8 PGA where $S_a(0.2)/PGA < 2$, or as equal to PGA, otherwise.

Step 3: From NBC Tables 4.1.8.4.-B to -F, determine values of F(0.2), F(5.0), $F(T_1)$ and $F(T_2)$ for the appropriate Site Class and for the value of PGA_{ref} determined in Step 2, using linear interpolation over PGA_{ref}.

Step 4: Using the values of F(0.2), F(5.0), $F(T_1)$ and $F(T_2)$ determined in Step 3, calculate S(0.2), S(5.0), $S(T_1)$ and $S(T_2)$ as follows:

$S(0.2) = F(0.2)S_a(0.2)$,
$S(5.0) = F(5.0)S_a(5.0)$,
$S(T_1) = F(T_1)S_a(T_1)$, and
$S(T_2) = F(T_2)S_a(T_2)$.

Then calculate the spectral ratio, S(0.2)/S(5.0).

Step 5: From NBC Table 4.1.8.11., determine values of $M_v(T_1)$, $M_v(T_2)$, $J(T_1)$ and $J(T_2)$ for the type of SFRS in the building and for the spectral ratio determined in Step 4, using linear interpolation over the spectral ratio.

Step 6: Using the values of $S(T_1)$ and $S(T_2)$ determined in Step 4 and the values of $M_v(T_1)$ and $M_v(T_2)$ determined in Step 5, calculate $S(T_1)M_v(T_1)$ and $S(T_2)M_v(T_2)$. Using these values, determine $S(T_a)M_v(T_a)$ by linear interpolation over the period. Similarly, using the values of $J(T_1)$ and $J(T_2)$ determined in Step 5, determine $J(T_a)$ by linear interpolation over the period. For moment-resisting frames, braced frames and other systems where $T_a \geq 2$ s, $S(T_a)M_v(T_a) = S(2.0)M_v(2.0)$ and $J(T_a) = J(2.0)$.

Sample Calculation of $S(T_a)M_v$ and J

167. This sample calculation illustrates how to calculate $S(T_a)M_V$ and J for a shear wall structure with $T_a = 1.5$ s located on stiff soil of Site Class D in Toronto.

Step 1: The period in NBC Table C-3 that is closest to and lower than T_a is $T_1 = 1.0$ s, and the period in the Table that is closest to and higher than T_a is $T_2 = 2.0$ s. From NBC Table C-3, the following values are obtained for Toronto:

$PGA = 0.160$ g,
$S_a(0.2) = 0.249$ g,
$S_a(1.0) = 0.063$ g,
$S_a(2.0) = 0.029$ g, and
$S_a(5.0) = 0.0071$ g.

Step 2: Because $S_a(0.2)/PGA = 0.249/0.160 = 1.56$, which is less than 2.0, PGA_{ref} is determined as follows:

$PGA_{ref} = 0.8 \times 0.160 = 0.128$ g

Step 3: From NBC Tables 4.1.8.4.-B, -D, -E and -F, values of F(0.2), F(1.0), F(2.0) and F(5.0) are obtained for PGA_{ref} values of 0.1 g and 0.2 g and for Site Class D. The F(T) values corresponding to $PGA_{ref} = 0.128$ g determined by interpolating between those for $PGA_{ref} = 0.1$ g and 0.2 g are:

F(0.2) = 1.1980,
F(1.0) = 1.5052,
F(2.0) = 1.5336, and
F(5.0) = 1.5520.

Step 4: Using the F(T) values determined in Step 3, the S(T) values are calculated as follows:

$S(0.2) = 1.1980 \times 0.249 = 0.2983$ g,
$S(1.0) = 1.5052 \times 0.063 = 0.0948$ g,
$S(2.0) = 1.5336 \times 0.029 = 0.0445$ g, and
$S(5.0) = 1.5520 \times 0.0071 = 0.0110$ g.

The spectral ratio is calculated as follows:

$$\frac{S\,(0.2)}{S\,(5.0)} = \frac{1.1980 \times 0.249}{1.5520 \times 0.0071} = 27.07$$

Step 5: From NBC Table 4.1.8.11., values of $M_v(1.0)$, $M_v(2.0)$, J(1.0) and J(2.0) are obtained for spectral ratios of 20 and 40. The values corresponding to S(0.2)/S(5.0) = 27.07 are determined by interpolating between those for S(0.2)/S(5.0) = 20 and S(0.2)/S(5.0) = 40 as follows:

$$M_v\,(1.0) = 1 + (27.07 - 20)\,\frac{(1.19 - 1)}{(40 - 20)} = 1.067$$

$$M_v\,(2.0) = 1.18 + (27.07 - 20)\,\frac{(1.75 - 1.18)}{(40 - 20)} = 1.381$$

$$J\,(1.0) = 0.80 + (27.07 - 20)\,\frac{(0.63 - 0.80)}{(40 - 20)} = 0.740$$

$$J\,(2.0) = 0.60 + (27.07 - 20)\,\frac{(0.46 - 0.60)}{(40 - 20)} = 0.551$$

Step 6: Using the values of S(1.0) and S(2.0) determined in Step 4 and the values of $M_v(1.0)$ and $M_v(2.0)$ determined in Step 5, $S(1.0)M_v(1.0)$ and $S(2.0)M_v(2.0)$ are calculated as follows:
$S(1.0)M_v(1.0) = 0.0948 \times 1.067 = 0.1012$ g, and
$S(2.0)M_v(2.0) = 0.0445 \times 1.381 = 0.0615$ g.

The value of $S(T_a)M_v(T_a)$ corresponding to $T_a = 1.5$ s is determined by interpolating between those for $T_a = 1.0$ and $T_a = 2.0$ as follows:

$$S\,(1.5)\,M_v\,(1.5) = 0.1012 + (1.5 - 1.0)\,\frac{(0.0615 - 0.1012)}{(2.0 - 1.0)} = 0.0814\text{ g}$$

The value of $J(T_a)$ corresponding to $T_a = 1.5$ s is determined by interpolating between those for $T_a = 1.0$ and $T_a = 2.0$ as follows:

$$J\,(1.5) = 0.740 + (1.5 - 1.0)\,\frac{(0.551 - 0.740)}{(2.0 - 1.0)} = 0.646$$

NBC Sentence 4.1.8.11.(7)

168. The ESFP approach involves both the calculation of the lateral earthquake design force, V, in accordance with NBC Sentence 4.1.8.11.(2), and the distribution of the force along the height of the building. For regular structures (i.e., those of uniform mass and storey height) where the dynamic response is primarily in the fundamental mode (which is the case for short-period structures), the response is very nearly proportional to height above the base of the structure. Since there is normally some variation in storey height and floor weight, NBC Sentence 4.1.8.11.(7) calls for the lateral force applied at each floor level x, F_x, to be proportional to floor weight at level x, W_x, multiplied by height above the base to level x, h_x, as follows:

$$F_x \propto W_x h_x / \sum_{i=1}^{n} W_i h_i$$

169. However, this inverted triangular load distribution fails to account for the effect of higher modes, which is significant for structures with medium to long fundamental lateral periods, even though the calculation of V takes this effect into account. Higher modes tend to increase the force in the upper storeys; this effect is taken into account in the distribution of load by specifying that a portion of the lateral earthquake design force, F_t, be applied as a concentrated load at the top of the building, and that the remainder of the force, V - F_t, be distributed as described in Paragraph 168. Accordingly, the lateral force applied to any floor level x, F_x, is calculated as follows:

$$F_x = (V - F_t)\, W_x h_x / \left(\sum_{i=1}^{n} W_i h_i \right)$$

The value of F_t depends on the fundamental lateral period, T_a, as follows:

$$\begin{aligned} F_t &= 0 && \text{for } T_a \leq 0.7\text{ s}, \\ F_t &= 0.07 T_a V && \text{for } 0.7\text{ s} < T_a < 3.6\text{ s, and} \\ F_t &= 0.25V && \text{for } T_a \geq 3.6\text{ s} \end{aligned}$$

Humar and Mahgoub[71] discuss the reasons why the distribution specified by NBC Sentence 4.1.8.11.(7) is adequate for design purposes.

NBC Sentence 4.1.8.11.(8)

170. As mentioned in Paragraph 165, NBC Table 4.1.8.11. gives values of the base overturning reduction factor, J, which are applied to overturning moments at the base of the structure to compensate for the overestimation of higher mode effects. A reduction factor must also be applied to overturning moments at other levels in the structure; NBC Sentence 4.1.8.11.(8) requires that the factor J_x for overturning moments at level x be calculated as follows:

$$\begin{aligned} J_x &= 1.0 \text{ for } h_x \geq 0.6 h_n, \text{ and} \\ J_x &= J + (1 - J)\,(h_x / 0.6 h_n) \text{ for } h_x < 0.6 h_n \end{aligned}$$

As indicated by the above expressions, no reduction is applied (i.e., $J_x = 1.0$) over the top 40% of the height of the building, and J_x decreases linearly over the bottom 60% of the height to J at the base. The factor J_x is applied as a multiplier to the overturning moment calculated at level x.

NBC Sentence 4.1.8.11.(9)

171. Although the lateral forces, F_x, defined in NBC Sentence 4.1.8.11.(7) are applied horizontally to a two-dimensional mathematical model of the structure, the actual structure is three-dimensional and responds both laterally and torsionally to earthquake ground motions. Observations during earthquakes have indicated that torsional vibrations are often the source of significant damage (Esteva[82] and Mitchell et al.[83]). NBC Sentence 4.1.8.11.(9) requires that torsional effects be considered concurrently with the effects of the lateral forces in the design of the structure. As indicated in NBC Article 4.1.8.8., earthquake effects are considered by performing independent analyses in two orthogonal horizontal directions; the two types of torsional moments described in NBC Clauses 4.1.8.11.(9)(a) and (b) must be included in each of the independent analyses.

172. Because the ESFP is by definition an elastic method of analysis, the torsional effects described in NBC Sentence 4.1.8.11.(9) are assumed to result from elastic behaviour. However, building structures subjected to DGMs at a probability of exceedance of 2% in 50 years are expected to behave inelastically. The effect of inelastic behaviour on lateral response is accounted for in a simplified manner by the application of the ductility-related force modification factor, R_d; there is no comparable methodology for accounting for the effect of inelastic behaviour on torsional response. This effect is considered by Humar et al.,[11] who show that, in most instances, the ductility demand at the edges of asymmetric buildings—the locations at which the deformations are the largest—is no greater than that in comparable symmetric or torsionally balanced buildings.

Clause (a): Torsional motion occurs in asymmetric structures, i.e., structures in which the centre of rigidity at each level of the structural system does not coincide with the centre of mass at that level. The centres of rigidity for the structural system are defined as the set of positions, one for each floor, at which the application of the full set of lateral forces results only in lateral deflection at all floor levels. When an asymmetric structure responds to a dynamic excitation, such as an earthquake ground motion, there may also be a dynamic amplification of the torsional motion.

Clause (b): Torsional motion may also occur in nominally symmetric structures with accidental eccentricities due to uncertainty in the determination of the centres of mass and rigidity, inaccuracy in the measured dimensions of structural elements, or variations in material properties, such as the modulus of elasticity. Another source of torsional vibration in nominally symmetric structures is rotational ground motion (Rutenberg and Heidebrecht[84]). The torsion resulting from all of these sources is referred to as accidental torsion.

NBC Sentence 4.1.8.11.(10)

173. Torsional effects can be considered using static analysis if the structure does not have torsional sensitivity, which is determined by calculating a torsional sensitivity parameter, B, as described in this Sentence. The calculation of B requires a static analysis using a three-dimensional elastic model of the SFRS in which the lateral force at each floor level is applied at distances of $\pm 0.10D_{nx}$ from the centre of mass, where D_{nx} is the plan dimension of the building at floor level x perpendicular to the direction of seismic loading being considered. The ratio, B_x, of the maximum lateral storey displacement at either of the two edges, δ_{max}, to the average of the displacements at the two edges, δ_{ave}, is calculated at each floor level x. The value of B for the entire building is the maximum of all of the values of B_x for both orthogonal horizontal directions of loading. The value of B_x for single-storey penthouses with a cumulative weight that is less than 10% of that of the level below need not be included because low-mass appendages have little effect on the overall torsional characteristics of a structure.

174. The determination of torsional sensitivity only applies to structures with rigid diaphragms, as indicated in NBC Table 4.1.8.6. Structures with flexible diaphragms are designed so that their loads, including the effects of accidental torsion, are distributed to the vertical elements using the tributary area concept. Accidental torsion should be taken into account by moving the centre of mass by $\pm 0.05D_{nx}$ and using the largest of the seismic loads for the design of each vertical element.

NBC Sentence 4.1.8.11.(11)

175. Torsionally stiff structures have small values of B, and torsionally flexible structures have large values of B. As shown by Humar et al.,[11] the static approach for the determination of torsional effects given in NBC Clause 4.1.8.11.(11)(a) is only valid for buildings that are relatively stiff in torsion, i.e., with $B \leq 1.7$. In buildings that are torsionally flexible, i.e., with $B > 1.7$, dynamic torsionally induced displacements cannot be reliably predicted using static measures of eccentricity. Since displacements in torsionally flexible structures are large and are therefore likely to be the source of significant distress, it is necessary to determine these displacements using a dynamic analysis that accounts for the effects of accidental eccentricity, as specified in NBC Clause 4.1.8.11.(11)(b).

Clause (a): In the ESFP, where $B \leq 1.7$ or $I_E F_a S_a(0.2) < 0.35$, torsional effects are accounted for by applying torsional moments, T_x, about the vertical axis at each level x of the building. For each direction of loading at each level x, the effects of these torsional moments, T_x, are applied in combination with the effects of the lateral force, F_x.

This Clause specifies that values of T_x are to be calculated separately for two load cases with different design eccentricities and that the elements of the building are to be designed for the most severe effect of the two load cases. Since the loads are to be applied in each of the two orthogonal horizontal directions, four distinct load cases must be considered.

The two design eccentricities, e_{d1} and e_{d2}, in each direction at each floor level x can be expressed as follows:

$$e_{d1} = e_x + 0.10D_{nx}, \text{ and}$$
$$e_{d2} = e_x - 0.10D_{nx}$$

where e_x is the natural eccentricity due to the centres of rigidity and mass being at different positions. De la Llera and Chopra[85] show that $\pm 0.05D_{nx}$ of the $\pm 0.10D_{nx}$ term represents accidental torsion; the remainder takes into account natural torsion, including dynamic amplification.

The combined lateral and torsional effects in each direction of loading can be determined by applying the lateral force at distances $+0.10D_{nx}$ and $-0.10D_{nx}$ from the centre of mass. The same set of load applications is used to determine the torsional sensitivity parameter, B (see NBC Sentence 4.1.8.11.(10)).

Clause (b): This Clause requires that the Dynamic Analysis Procedure, as specified in NBC Article 4.1.8.12., be used in cases where $B > 1.7$ and $I_E F_a S_a(0.2) \geq 0.35$ because the ESFP does not adequately take into account the potential for large displacements in such cases (Humar et al.[11]). However, as indicated in NBC Clause 4.1.8.7.(1)(a), the ESFP may be used in regions of low seismicity regardless of whether torsional sensitivity or any other permissible irregularity exists; the approximations inherent in the ESFP are unlikely to have serious consequences for the relatively small ground motions predicted for such regions.

NBC Sentence 4.1.8.11.(12)

176. For buildings constructed with more than 4 storeys of continuous wood construction, a two-dimensional study undertaken by the APEGBC Six Storey Wood Frame Building Structural Task Force[86] recommends that, where the fundamental lateral period, T_a, is determined using an established method of mechanics rather than an empirical formula, the lateral earthquake design force, V, determined by the ESFP be increased by 20% to reduce the risk of a weak storey forming. NBC Sentences 4.1.8.11.(12) and 4.1.8.12.(12) are based on the findings of this study. See Paragraph 153 for information on determining the number of storeys for the application of these Sentences.

Dynamic Analysis Procedure (NBC Article 4.1.8.12.)

177. As indicated in NBC Article 4.1.8.7., dynamic analysis is mandatory for the determination of earthquake design actions, except in situations where the ESFP is adequate.

178. Linear Dynamic Analysis must be conducted in accordance with the procedures of NBC Article 4.1.8.12. by performing analyses using two different models: one in which all lateral displacements other than those in the direction of the earthquake forces are restrained and the floor and roof rotations about a vertical axis are restrained, and the other in which the floor and roof rotations are unrestrained. The first analysis is used to determine the scale factor that must be applied to the results of the second analysis. The following steps are involved in a Linear Dynamic Analysis procedure:

Step 1: Construct a structural model of the building taking into account the requirements in NBC Sentence 4.1.8.3.(8).

Step 2: Using the model from Step 1 with all lateral displacements other than those in the direction of the earthquake forces restrained, carry out a linear dynamic analysis to determine the fundamental lateral period, T_a, and the elastic base shear (also referred to as the "lateral earthquake elastic force"), V_e.

Step 3: If NBC Sentence 4.1.8.12.(6) applies, determine the following factors using the value of T_a from Step 2:

$$\frac{2S(0.2)}{3S(T_a)} \leq 1.0, \text{ and}$$

$$\frac{S(0.5)}{S(T_a)} \leq 1.0$$

Multiply the larger of the two factors by the value of V_e from Step 2 to determine the design elastic base shear (also referred to as the "lateral earthquake design elastic force"), V_{ed}.

Step 4: Determine the design base shear (also referred to as the "lateral earthquake design force"), V, by the ESFP given in NBC Article 4.1.8.11. In determining V, T_a may be taken as the smaller of the value of T_a obtained in Step 2 and the applicable upper limit on T_a specified in NBC Subclauses 4.1.8.11.(3)(d)(i) to (d)(iv). In calculating the scale factor to be applied to deflections in Step 5, V may be determined using the smaller of the value of T_a obtained in Step 2 and the applicable upper limit on T_a specified in NBC Subclause 4.1.8.11.(3)(d)(v).

If required for the calculation of the effects of accidental torsion according to NBC Sentence 4.1.8.12.(4), determine the lateral forces, F_x, at each floor level x by distributing V along the height of the structure. If the torsional sensitivity parameter, B, has not already been determined, calculate its value using these values of F_x.

Step 5: Using the value of V_{ed} from Step 3, obtain the design base shear, V_d, in accordance with NBC Sentences 4.1.8.12.(7) to (9) and (12). Note that the value of V_d can be no less than 0.8V for regular structures and irregular structures permitted to be designed using the ESFP, and no less than V for irregular structures requiring dynamic analysis and for buildings constructed with more than 4 storeys of continuous wood construction satisfying the requirements of NBC Sentence 4.1.8.12.(12), where V is the design base shear calculated in Step 4. Determine the scale factor, V_d/V_e, using the value of V_e obtained in Step 2. This scale factor will be applied to the elastic storey shears, storey forces, member forces and deflections obtained from a dynamic analysis of the model in which the floors and roofs are unrestrained.

Step 6: If accidental torsion is to be accounted for using the procedure specified in NBC Clause 4.1.8.12.(4)(a), carry out a three-dimensional elastic linear dynamic analysis on the model constructed in Step 1 to obtain the elastic storey shears, storey forces, member forces and deflections; otherwise, go to Step 8.

Step 7: Calculate the effects of accidental torsion in accordance with NBC Clause 4.1.8.12.(4)(a) and add them to the effects determined in Step 6 to obtain the elastic storey shears, storey forces, member forces and deflections including the effects of accidental torsion. The lateral forces, F_x, at each floor level x required for the calculation of the static effects may be taken as the forces determined in Step 4 multiplied by R_dR_o/I_E. Alternatively, they may be obtained from the storey shears determined in Step 6 by calculating F_x as the difference between the maximum dynamic shear in the storey below level x and that in the storey above level x.

Step 8: If accidental torsion is to be accounted for using the procedure specified in NBC Clause 4.1.8.12.(4)(b), carry out two separate three-dimensional elastic linear dynamic analyses: one using the model constructed in Step 1 with the centres of mass shifted by $-0.05D_{nx}$, and the other using the same model with the centres of mass shifted by $+0.05D_{nx}$ (the centres of mass are shifted in the same direction for all storeys). The larger of the values obtained from the two analyses provides the elastic storey shears, storey forces, member forces and deflections, including the effects of accidental torsion.

Step 9: Scale the storey shears, storey forces, member forces and deflections obtained in Step 7 or Step 8 using the scale factor obtained in Step 5. Note that the deflections are elastic and need to be multiplied by R_dR_o.

NBC Sentence 4.1.8.12.(1)

179. This Sentence indicates that it is permissible to do either a Linear or a Non-linear Dynamic Analysis.

Clause (a): A Linear Dynamic Analysis, using either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method, is the normal approach because the analysis procedures are straightforward and can be found in texts on structural dynamics (e.g., Chopra[87] and Humar[88]). Also, standard software used for structural analysis often includes Linear Dynamic Analysis as one of the options; the Modal Response Spectrum Method is more commonly included in such software. The structural model used in Linear Dynamic Analysis must comply with the requirements of NBC Sentence 4.1.8.3.(8) to ensure that it represents the actual structure in a realistic manner. The other Sentences in NBC Article 4.1.8.12. prescribe how the dynamic excitation is to be determined and how the results are to be used in design, including how accidental torsion is to be taken into account. Saatcioglu and Humar[12] discuss some of these requirements, including considerations such as the number of modes required to accurately represent the dynamic response of the structure.

The Modal Response Spectrum Method is based on the fact that the response of a linear elastic system is made up of the superposition of the responses of individual natural modes of vibration, each mode responding at its natural frequency with its own pattern of deformation, i.e., its mode shape. The most common form of this method involves the combination of the maximum response parameters in each mode to determine the maximum values of the response parameters for the structure as a whole. Only a small number of modes—around 3 to 5—are required to provide a good approximation of the total response; Chopra[87] discusses the factors involved in selecting the number of modes, including the desired accuracy and the response quantity of interest. FEMA 368, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions," provides a simple rule for the determination of the number of modes required: the normal requirement is that the combined participating mass of all the modes included in the analysis should total at least 90% of the total mass. The primary sources of uncertainty in this method are the validity of the structural model, the validity of the modal combination rule, and the value of damping in each mode.

The Numerical Integration Linear Time History Method involves the determination of the response of a structural model to a specific earthquake ground motion accelerogram through the numerical integration of the equations of motion. The primary advantage of this method, compared with the Modal Response Spectrum Method, is that the various response parameters are obtained as time histories, providing information on the time-wise fluctuation of the state of deformation of the structure. There are several disadvantages to using this method; most notably, it produces voluminous amounts of data to be interpreted, and the results depend greatly on the characteristics of the individual ground motion accelerograms so the analysis needs to include a number of different time histories. Because of these disadvantages and the resulting increased costs of analysis, this method is rarely used for the design of ordinary building structures.

The selection and scaling of ground motions for this method must be performed as described in the Commentary section on NBC Clause 4.1.8.12.(1)(b). The section titled Design Seismic Demand in the Appendix to this Commentary defines the seismic demand values obtained from analysis that must be used for design. Interstorey drifts obtained from analysis must not exceed the limits specified in NBC Sentence 4.1.8.13.(3). The values of the forces to be used for the design of the SFRS elements depend on whether the forces are designated as deformation-controlled or force-controlled actions, as defined in item (viii) of the Commentary section on NBC Clause 4.1.8.12.(1)(b). Forces designated as deformation-controlled actions that are obtained from analysis must be multiplied by I_E/R_dR_o to obtain design forces. Forces designated as force-controlled actions must be obtained from capacity design principles, i.e., taken as equal to the forces induced by gravity loads plus the forces induced by lateral loads corresponding to the probable lateral resistance of the SFRS, as specified in the applicable CSA standard. Forces designated as force-controlled actions obtained from analysis are not permitted to be used in design.

Clause (b): A Non-linear Dynamic Analysis is an acceptable alternative to a linear one, provided that a special study is performed. Since such analyses are still primarily done in a research environment, it is essential that the special study be conducted and peer-reviewed by individuals who are competent and experienced in making the necessary judgments and decisions. In addition, the resulting design should be reviewed by a qualified independent engineering team. Where non-linear analysis is used to determine the seismic response of a structure, all of the general and specific requirements of NBC Subsection 4.1.8. apply nevertheless. Particular attention must be given to the requirements for stiff elements (NBC Sentences 4.1.8.3.(6) and (7)), the effect of site classification on ground motion values (NBC Article 4.1.8.4.), the use of an appropriate earthquake importance factor (NBC Article 4.1.8.5.), and the restrictions on structural configuration (NBC Article 4.1.8.6.). The resulting design will, in most instances, be expected to have features (e.g., member sizes and stiffnesses) that are similar to those of a design obtained using Linear Dynamic Analysis or the ESFP.

The following considerations are of particular importance in the special study:

(i) Independent design review is required when non-linear time history analysis is used. The review must be performed by a peer review panel of at least three reviewers, including at least one reviewer having recognized expertise in each of the following areas: non-linear time history analysis, earthquake-resistant design, and seismic hazard.

(ii) The ground motion time histories used as input should be representative of the seismotectonic environment and the geotechnical conditions at the location of the building, and should be selected and scaled according to the guidelines in the Appendix to this Commentary. These guidelines are based on the provisions proposed for ASCE/SEI 7-16 (Haselton et al.[89]), but include several differences that reflect the provisions of the NBC.

The ground motion records selected must cover the range of periods that contribute significantly to the seismic response of the building, i.e., the period range of interest, T_R, as explained in the section titled Period Range, T_R, in the Appendix to this Commentary. Typically, suites of ground motion records are selected to cover two or more segments of the period range of interest by considering earthquakes associated with different dominant magnitude–distance scenarios or earthquakes from different sources or tectonic environments (e.g., shallow crustal, subduction interface and subduction intraslab in southwestern British Columbia) that contribute to the seismic hazard at the site, as revealed by site-specific seismic hazard disaggregation. Each of these period segments constitutes a scenario-specific period range, T_{RS}; together, they must cover the entire period range of interest, T_R.

The ground motion records selected for each suite must be representative of the magnitude–distance scenario and the tectonic environment associated with the T_{RS}. It is recommended that a minimum of 11 ground motion records be used for each suite. Using fewer than 11 records for a suite is permitted, provided that no less than five records are used for each suite, the number of records is approved by the review panel, and the total number of records in all the suites is not less than 11. For example, where earthquakes from only one tectonic environment contribute to the seismic hazard at the site, the number of records per suite can be reduced to five, but the total number of records in all the suites must be at least 11. These numbers of records are generally sufficient to provide a mean seismic demand that is consistent with the target spectrum.

For structures with seismic isolation or a supplemental energy dissipation system, the seismic demand is not determined by simply taking the mean values of forces and deflections (see the section titled Design Seismic Demand in the Appendix to this Commentary). A much larger number of ground motion records—typically more than 30—is needed to characterize the dispersion in the seismic response of such structures. The response spectra of the ground motion records selected for each suite should match the target spectrum for the scenario-specific period range, T_{RS}.

In Method A of the Appendix to this Commentary, the design response spectrum, with modifications to better reflect the seismic demand in the short-period range, is used as the target spectrum (see the Commentary section on NBC Sentence 4.1.8.12.(2) starting at Paragraph 180).

Alternatively, in Methods B1 and B2 of the Appendix to this Commentary, site-specific scenario target spectra may be developed for each scenario-specific period range, T_{RS}, and used as the target spectrum (see the section titled Horizontal-Component Target Spectrum in the Appendix to this Commentary). Recorded ground motion time histories from past earthquakes are generally preferred; however, simulated ground motion time histories can be used in the absence of representative recorded ones. The section titled Scaling of Ground Motions in the Appendix to this Commentary provides a two-stage procedure and criteria for scaling the ground motion records with respect to the target spectrum; guidance is also given on scaling vertical ground motion components and pairs of orthogonal horizontal ground motion components.

Examples of the selection and scaling of ground motion time histories according to the guidelines in the Appendix to this Commentary are given by Tremblay et al.[90] Additional information on the selection and scaling of ground motion time histories can be found in Haselton et al.,[91] NEHRP,[92] Baker,[93] and Daneshvar et al.[94] Information on seismicity in Canada and the assessment of seismic hazard can be found in Atkinson and Adams,[13] Halchuck et al.,[14][27][28] and Rogers et al.[15]

Ground motion time histories are available in several databases. The Engineering Seismology Toolbox (www.seismotoolbox.ca) contains simulated ground motion records for Site Classes A, C, D and E for both western and eastern seismic regions of Canada, as well as predicted ground motions for large subduction earthquakes anticipated in the Cascadia subduction zone (Atkinson[95]). The Pacific Earthquake Engineering Research Center's (PEER) NGA-West2 ground motion database (ngawest2.berkeley.edu) contains a large number of ground motions recorded during shallow crustal earthquakes in active tectonic regimes (Ancheta et al.[96]). The PEER NGA-East ground motion database (ngawest2.berkeley.edu) contains ground motion records for central and eastern North America (Goulet et al.[97] and PEER[98]).

Other available databases include the Consortium of Organizations for Strong Motion Observation Systems' (COSMOS) Strong Motion Data Center (strongmotioncenter.org/vdc/scripts/default.plx) and the National Research Institute for Earth Science and Disaster Resilience (NIED) of Japan's K-NET and KiK-net databases (www.kyoshin.bosai.go.jp).

(iii) The characteristics of the non-linear cyclic response of the structural elements in the model (e.g., strength, stiffness, ductility capacity and hysteretic behaviour) must be representative of the behaviour of actual elements that have been subjected to reversed cyclic loading tests in the non-linear range. While the modeling of flexural elements with well-defined yielding and hysteretic behaviour is relatively easy, the modeling of structural elements with other types of non-linear behaviour (e.g., connections exhibiting degradation of strength and stiffness, and bracing members subjected to reversal of inelastic buckling and tension yielding) is more complex, particularly if their failure modes are brittle or minimally ductile. Saatcioglu and Humar[12] present information on hysteretic models that are commonly used to represent structural elements.

(iv) The strength properties used to model the non-linear elements must be determined in accordance with the requirements of the applicable CSA standard. Where there is no applicable CSA standard, they must be determined in consultation with the peer review panel.

In determining displacement demands, and forces and deformations designated as deformation-controlled actions, the following approach must be followed:
- lower-bound strength values must be used to model the non-linear SFRS elements;
- unless a higher value can be justified, the lower-bound strength value must be taken as 1.1 times the nominal strength (nominal resistance with a resistance factor, φ, of 1);
- where explicitly modeled in the analysis, the deformation capacities of non-linear SFRS elements should not exceed the values specified for deformation-controlled actions in item (ix); and
- where strain hardening is expected, an appropriate strain-hardening response should be included in the model.

In determining forces designated as force-controlled actions in accordance with item (x), the following approach must be followed:
- upper-bound strength values must be used to model the non-linear SFRS elements;
- unless a lower value can be justified, the upper-bound strength value must be taken as 1.2 times the probable resistance as specified in the applicable CSA standard; and
- strain-hardening effects must be included in the model.

(v) The inherent damping of the structure, which is not associated with the response of non-linear elements, is permitted to be included in the model in addition to the hysteretic energy dissipation capacity of the non-linear elements. Unless a higher value can be justified, the inherent damping of the structure should not exceed 3% of the critical damping in the modes dominating the seismic response of the structure. Geometric non-linearities (P-delta effects) must also be taken into account in the analysis by using the concomitant gravity loads, expressed as load combinations in accordance with NBC Article 4.1.3.2. Where the analysis is performed using a three-dimensional structural model with pairs of orthogonal horizontal ground motion components, the distribution of mass in the model should reflect the actual conditions, and accidental eccentricity must be considered independently in each orthogonal horizontal direction by displacing the centre of mass by 5% of the building dimension perpendicular to the direction under consideration.

(vi) The interpretation of the results of the analysis for the design of members must take into account both global (e.g., lateral displacements and interstorey drifts) and local force and deformation demands on the SFRS elements. The section titled Design Seismic Demand in the Appendix to this Commentary defines the seismic demand values obtained from analysis that must be used for design.

(vii) Interstorey drifts can be taken as the design demand values obtained from an analysis using lower-bound strength values as defined in item (iv), but they must not exceed the limits specified in NBC Sentence 4.1.8.13.(3).

(viii) Actions in SFRS elements must be classified as either deformation-controlled or force-controlled according to an approach similar to the one adopted in ASCE/SEI 41. Deformation-controlled actions in SFRS elements are deformations and forces associated with a ductile non-linear response under reversed cyclic loading: e.g., ductile plastic rotation in beams of ductile steel moment-resisting frames, and flexural yielding at the base of ductile reinforced concrete shear walls. Deformation-controlled actions are permitted in SFRS elements that are specifically designed and detailed in accordance with the applicable CSA standard to exhibit a ductile non-linear response under reversed cyclic loading, as well as in those for which a satisfactory ductile non-linear response under reversed cyclic loading has been demonstrated through physical testing. Forces in SFRS elements that are not considered to be deformation-controlled actions are designated as force-controlled actions: e.g., axial loads in columns and shears in reinforced concrete shear walls.

(ix) Deformations in SFRS elements that are designated as deformation-controlled actions must be taken as the design demand values obtained from an analysis using lower-bound strength properties as defined in item (iv). These deformations must not exceed the limits specified in the applicable CSA standard multiplied by $1.0/I_E$. In the absence of such limits, the deformations must not exceed the acceptance criteria for the Life Safety Structural Performance Level as defined in ASCE/SEI 41 multiplied by $0.7/I_E$. Lower-bound strength values as defined in item (iv) must be used with ASCE/SEI 41 to determine the deformation capacities that depend on strength. Alternatively, in the absence of limits specified in the applicable CSA standard, test data may be used to determine the deformation limits; in such cases, the deformation limit of an element must be taken as $0.5/I_E$ times the mean of the deformation values causing failure of the element, unless a higher deformation limit can be justified to the peer review panel.

(x) Forces in SFRS elements that are designated as force-controlled actions must be determined according to capacity design principles, i.e., taken as the forces induced by gravity loads plus the forces induced by lateral loads corresponding to the probable lateral resistance of the SFRS as specified in the applicable CSA standard. Forces designated as force-controlled actions may be taken as the design demand values obtained from an analysis using upper-bound strength values as defined in item (iv).

NBC Sentence 4.1.8.12.(2)

180. The Modal Response Spectrum Method requires that the dynamic excitation be represented as an acceleration response spectrum, i.e., the maximum acceleration response of a single-degree-of-freedom system with varying period when subjected to a specific ground motion time history. NBC Sentence 4.1.8.12.(2) specifies that the spectral response acceleration values, i.e., the ordinates of the acceleration response spectrum, be the design spectral response acceleration values, S(T), as defined in NBC Sentence 4.1.8.4.(9), which include site effects through the use of the site coefficients, F(T).

181. In contrast, the Numerical Integration Linear Time History Method and the Non-linear Dynamic Analysis Method require the selection of realistic ground motion time histories that are compatible with the uniform hazard spectrum for the site. For this purpose, the design response spectrum can be defined by the S(T) values at periods of 0 s, 0.05 s, 0.1 s and 0.3 s in addition to the S(T) values specified in NBC Sentence 4.1.8.4.(9), and need not have an upper plateau at $T \leq 0.2$ s. The value of S(0) is equal to the value of PGA modified by the corresponding site coefficient, F(PGA) (see the section titled Design Spectrum, S(T), in the Appendix to this Commentary). Values of spectral response acceleration, $S_a(T)$, at periods of 0.05 s, 0.1 s and 0.3 s are not specified in NBC Table C-3, but can be obtained from the Geological Survey of Canada (by using the “Hazard Calculator” on the Earthquakes Canada Web site). Values of the corresponding site coefficients, F(0.05), F(0.1) and F(0.3), are given in Table J-4. A design response spectrum containing the S(T) values for the additional periods will better represent higher mode effects than the simplified spectrum constructed from only the S(T) values specified in NBC Sentence 4.1.8.4.(9). In realistic response spectra, the spectral response acceleration decreases to PGA as the period approaches zero, and there is no upper plateau as specified in NBC Sentence 4.1.8.4.(9). The adjustment of ground motion time histories to be compatible with a design response spectrum having an upper plateau is, therefore, likely to lead to unrealistic results.

As indicated in Note (2) to NBC Table 4.1.8.4.-A, where $\bar{V}_{s30}$ has been measured in situ, the F(0.3) values for Site Class A derived from Table J-4 may be multiplied by the factor $0.04 + (1\,500/\bar{V}_{s30})^{1/2}$. However, this adjustment should not be applied to F(0.05) or F(0.1) values derived from the Table.

In general, rock and hard rock sites experience lower ground motion amplitudes than sites with softer soils. However, the F(0.05) values for Site Class A in Table J-4 are greater than 1, signifying an amplification of ground motions, because it has been observed that hard rock sites in eastern Canada allow more short-period energy to travel to the surface instead of being attenuated or scattered. This effect is less pronounced for PGA because its short-period energy content is less dominant.

Table J-4
Values of F(0.05), F(0.1) and F(0.3) as a Function of Site Class and PGA_{ref}

Site Class	Values of F(0.05)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	1.02	1.02	1.02	1.02	1.02
B	0.94	0.94	0.94	0.94	0.94
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.05	0.96	0.89	0.85
E	1.65	1.11	0.88	0.74	0.65
F	(1)	(1)	(1)	(1)	(1)
	Values of F(0.1)				
A	0.83	0.83	0.83	0.83	0.83
B	0.85	0.85	0.85	0.85	0.85
C	1.00	1.00	1.00	1.00	1.00
D	1.21	1.04	0.94	0.89	0.84
E	1.55	1.08	0.87	0.75	0.67
F	(1)	(1)	(1)	(1)	(1)
	Values of F(0.3)				
A	0.62	0.62	0.62	0.62	0.62
B	0.70	0.70	0.70	0.70	0.70
C	1.00	1.00	1.00	1.00	1.00
D	1.34	1.17	1.08	1.02	0.98
E	1.97	1.46	1.22	1.08	0.97
F	(1)	(1)	(1)	(1)	(1)

(1) Site-specific evaluation is required to determine F(T) for Site Class F.

NBC Sentence 4.1.8.12.(3)

182. As noted in the Commentary section titled Seismic Hazard (starting at Paragraph 38), the design response spectrum constructed from S(T) values is essentially a uniform hazard spectrum, i.e., a plot of spectral response acceleration ordinates at different periods, each ordinate having the same probability of exceedance. While this design response spectrum is not an acceleration response spectrum from an individual earthquake, Humar and Mahgoub[71] show that the uniform hazard spectrum is a slightly conservative representation of an actual response spectrum, but that its use results in no more than a 10% overestimation of the response of a multi-degree-of-freedom structure.

NBC Sentence 4.1.8.12.(4)

183. Three-dimensional analysis provides a good representation of the behaviour of structures with torsional eccentricity and, in accordance with NBC Clause 4.1.8.11.(11)(b), is required for torsionally sensitive structures, i.e., for which the torsional sensitivity parameter, B, is greater than 1.7. Such analysis should include the mass moments of inertia of the floors. However, structural modeling for such an analysis does not ordinarily take into account the effects of accidental eccentricities, which must be considered to act concurrently with the effects of lateral motion including actual eccentricities. NBC Sentence 4.1.8.12.(4) provides alternative approaches for determining the effects of accidental eccentricity, the second of which is only permitted where $B < 1.7$.

Clause (a): In the first approach, which can be used for any value of B but is intended primarily for torsionally sensitive structures, the effects of static torsional moments, $(\pm 0.10D_{nx})F_x$, at each level x are calculated and then combined with the effects determined from a dynamic analysis that includes the actual eccentricities. The lateral forces, F_x, may be either those determined from a static analysis (as specified in NBC Sentence 4.1.8.11.(7)) multiplied by R_dR_o/I_E, or those determined from a dynamic analysis in which the floors and roofs are allowed to translate as well as rotate. Where a dynamic analysis is used, the lateral force at level x, F_x, may be taken as the difference between the maximum dynamic shear in the storey below level x and that in the storey above level x. As discussed in the Commentary section on NBC Sentence 4.1.8.11.(11) (see Paragraph 175), the accidental eccentricity is represented by $0.05D_{nx}$; the value of $0.10D_{nx}$ stated in this Clause includes a dynamic amplification of the static effect of accidental eccentricity.

Clause (b): The second approach is only permissible for structures that are not torsionally sensitive, i.e., for $B < 1.7$. This approach allows the effects of accidental eccentricity to be included by shifting the centres of mass by $\pm 0.05D_{nx}$. Two three-dimensional dynamic analyses are therefore required—one for each of the two locations of the shifted centre of mass. The larger of the two values of V_e must be used for determining the design elastic base shear, V_{ed}, in accordance with the provisions of NBC Sentence 4.1.8.12.(5). For any effect, the larger of the values obtained from the two dynamic analyses must be used to determine the design value.

NBC Sentence 4.1.8.12.(5)

184. For short-period structures that have some ductility capacity and are not located on very poor soil, the ESFP specifies an upper limit on the design base shear. For consistency, a similar provision exists when a dynamic analysis is used to determine the design forces. NBC Sentence 4.1.8.12.(6) specifies a procedure for adjusting the elastic base shear, V_e, obtained from linear dynamic analysis to in turn obtain the design elastic base shear, V_{ed}, for short-period structures. For cases not covered by that Sentence, V_{ed} is equal to V_e. For the reason outlined in the Commentary section on NBC Sentence 4.1.8.12.(9) (starting at Paragraph 189), V_e is determined from a model in which the floors and roofs are restrained so that there is motion only in the direction of earthquake forces. Accordingly, values of V_{ed} and V_d referenced in NBC Sentences 4.1.8.12.(5) to (12) are obtained from a dynamic analysis on a model in which floor and roof rotations are restrained. These values of V_e and V_d are only used for determining the scale factor to be applied to the storey shears, storey forces, member forces and displacements obtained from an analysis of a model in which the floors and roofs are allowed to rotate.

NBC Sentence 4.1.8.12.(6)

185. For the reasons described in the Commentary section on NBC Sentence 4.1.8.11.(2) (see Paragraph 157), a reduction factor is applied to the elastic base shear for short-period structures having a value of $R_d \geq 1.5$ and located on sites other than Class F to limit the design forces for such structures. The elastic base shear, V_e, is multiplied by the larger of the following two factors to obtain the design elastic base shear, V_{ed}:

$$\frac{2S(0.2)}{3S(T_a)} \leq 1.0, \text{ and}$$
$$\frac{S(0.5)}{S(T_a)} \leq 1.0$$

Application of the reduction factor to V_e limits the design spectral response acceleration at the fundamental (first mode) lateral period to the larger of 2/3S(0.2) and S(0.5); the spectral response accelerations for higher modes are automatically reduced by the same factor so that the relative contributions of the various modes remain unchanged.

NBC Sentence 4.1.8.12.(7)

186. The design elastic base shear, V_{ed}, does not take into account either the inelastic response or the Importance Category of the structure. Consequently, in order to determine the design base shear, V_d, the design elastic base shear must be divided by the product R_dR_o and multiplied by the earthquake importance factor, I_E. By making these adjustments, V_d is determined on a comparable basis to the static design base shear, V, of NBC Sentence 4.1.8.11.(2). Note that the upper bound on V specified in NBC Clause 4.1.8.11.(2)(c) is applied to V_d through the provision of NBC Sentence 4.1.8.12.(6), while the lower bounds on V specified in NBC Clauses 4.1.8.11.(2)(a) and (b) are applied directly through the shape of the design response spectrum specified in NBC Sentence 4.1.8.4.(9), and indirectly by setting the lower bound on the dynamic design base shear to 0.8V or V in accordance with NBC Sentence 4.1.8.12.(8), (9) or (12), as applicable.

NBC Sentence 4.1.8.12.(8)

187. If the modeling of the structure is done correctly, the design base shear determined from a Linear Dynamic Analysis, V_d, will be a more accurate representation of the behaviour of the structure than the design base shear determined by the ESFP, V. However, structural models tend to be more flexible than actual structures, one reason being that they do not take into account stiff non-structural elements. Because the design spectral response acceleration decreases with increasing flexibility (i.e., increasing period), there is concern that this tendency will result in V_d being less than it should be. NBC Sentence 4.1.8.12.(8) addresses this concern by requiring that V_d be taken as 0.8V when the calculated value of V_d is less than 80% of the value of V determined in NBC Article 4.1.8.11. A reduction of up to 20% of V is deemed reasonable because dynamic analysis results in a better distribution of forces within the structure. Of course, where $V_d \geq 0.8V$, the calculated value of V_d must be used as the design base shear because the value of V_d obtained from dynamic analysis is expected to be more accurate than the value of V obtained from static analysis. The value of V_d is expected to be greater than the value of V in situations where the dynamic model is stiffer than the static model (resulting in a smaller value of the fundamental lateral period) or where higher modes dominate the dynamic response (e.g., in tall, flexible structures with long periods or in cases where there is a flexible, long-period tower on top of a large, heavy, short-period podium).

188. The value of V used to determine the minimum value of V_d can be calculated using a fundamental lateral period determined according to an established method of mechanics rather than from an empirical formula, as permitted in NBC Clause 4.1.8.11.(3)(d), provided that this period does not exceed the specified limits. It is also acceptable to determine V using the fundamental lateral period calculated from the structural model used for dynamic analysis. In this case, the only significant difference between the static and dynamic approaches is that the latter one considers the higher modes and their effects on the distribution of forces and deflections along the height of the structure.

NBC Sentence 4.1.8.12.(9)

189. Taking the design base shear as 0.8V is not permitted in situations where dynamic analysis is required to account for the irregularity of the structure according to NBC Article 4.1.8.7. In such cases, the modeling of the structure for dynamic analysis may not fully capture the influence of irregularities on its behaviour during an earthquake, particularly since the actual structure will behave in an inelastic manner, most likely with concentrations of inelastic demand at points of stiffness or mass discontinuity. Consequently, when the presence of irregularities results in dynamic analysis being required, the minimum value of V_d used for design must be V or the calculated value of V_d, whichever is greater.

190. NBC Sentences 4.1.8.12.(8), (9) and (12) require that the design base shear, V_d, be not less than 0.8V or V. When the elastic base shear, V_e, is determined from a three-dimensional analysis of a torsionally eccentric structure, the coupling of lateral and torsional responses can produce a value of V_e that is considerably lower than that for a comparable torsionally balanced structure, i.e., one having the same characteristics but with coincident centres of mass and resistance. Therefore, the requirement that V_d be not less than 0.8V or V would be overly conservative and would require that a large scale factor be applied to the results. A method of determining the scale factor that is more consistent with the intent of NBC Sentences 4.1.8.12.(8), (9) and (12) is to carry out an analysis on a model in which the rotations of the floors and roofs are restrained so that there is motion in only one direction, and to use the resulting V_d to calculate the scale factor. This scale factor can then be applied to the design base shear, V_d, and the member forces and displacements determined from the dynamic analysis of a model in which the floors and roofs are allowed to rotate.

NBC Sentence 4.1.8.12.(10)

191. NBC Sentences 4.1.8.12.(5) to (7) specify how the design base shear, V_d, is to be determined from the elastic base shear, V_e. Although these Sentences address the determination of the design base shear, it is important that all of the other design actions, e.g., element forces, storey shears and interstorey drifts, including the effects of accidental torsion determined in NBC Sentence 4.1.8.12.(4), be proportioned accordingly. Since the initial determination of these actions is associated with V_e, NBC Sentence 4.1.8.12.(10) requires that the design actions be calculated by multiplying the initial actions by the ratio V_d/V_e. It should be noted that the resulting design deflections and interstorey drifts are elastic and need to be multiplied by the product R_dR_o/I_E in order to obtain realistic values of anticipated deflections and drifts, as specified in NBC Sentence 4.1.8.13.(2).

NBC Sentence 4.1.8.12.(12)

192. A study undertaken by the APEGBC Six Storey Wood Frame Building Structural Task Force[86] states that, where the fundamental lateral period of a building constructed with more than 4 storeys of continuous wood construction is determined using an established method of mechanics rather than an empirical formula, the design base shear determined using a Dynamic Analysis Procedure must not be less than 100% of the design base shear determined using the ESFP to reduce the risk of a weak storey developing. Accordingly, NBC Sentence 4.1.8.12.(12) requires that the design base shear, V_d, be taken as the larger of the value of V_d determined by dynamic analysis and 100% of the value of V determined by static analysis in accordance with NBC Article 4.1.8.11. See the Commentary sections on NBC Sentences 4.1.8.10.(4) (Paragraph 152) and 4.1.8.11.(12) (Paragraph 176) for more information on buildings constructed with more than 4 storeys of continuous wood construction.

Deflections and Drift Limits (NBC Article 4.1.8.13.)

193. The damage caused to buildings by earthquake ground motions is a direct consequence of the lateral deflection of the structural system. The ability of a building to withstand such ground motions arises largely from the capability of the structural system to deform without significant loss of load-carrying capacity. NBC Article 4.1.8.13. is concerned with both the determination of lateral deflections and the placement of limits on those deflections to ensure satisfactory performance. In this context, lateral deflections are relative to the ground, i.e., the top of the foundation at the base of the structure.

NBC Sentence 4.1.8.13.(1)

194. This Sentence requires that the loads and other requirements defined in NBC Subsection 4.1.8. be used in the calculation of lateral deflections. Static loads are defined in NBC Article 4.1.8.11., and dynamic loads are defined in NBC Article 4.1.8.12. The next most important requirement is that the structural modeling be representative of the actual building structure and account for the specific features and effects listed in NBC Sentence 4.1.8.3.(8). Although the stiffness of elements that are not part of the SFRS is to be accounted for in determining the period of the structure where the added stiffness decreases the fundamental lateral period by more than 15% (see NBC Sentence 4.1.8.3.(7)), such elements should not be included in the modeling of the structure for the purpose of calculating lateral deflections. Stiff elements that are not part of the SFRS are likely to crack and lose their stiffness as the structure responds to strong earthquake ground motions; they are, therefore, unlikely to participate in limiting the lateral deflections of the structure. Only the structural elements that are part of the SFRS should be used in the determination of lateral deflections.

NBC Sentence 4.1.8.13.(2)

195. As explained in Mitchell et al.,[9] the lateral earthquake design force (design base shear) is reduced to take into account inelastic behaviour and overstrength. The lateral deflections calculated from the reduced design force are elastic deflections, which do not include incursions into the inelastic range, rather than maximum deflections, which do. As shown in Figure 2 of Mitchell et al.,[9] for I_E = 1.0, the maximum deflection is R_dR_o times the deflection determined using the lateral earthquake design force, V, specified in NBC Sentence 4.1.8.11.(2). Accordingly, NBC Sentence 4.1.8.13.(2) requires that the lateral deflections determined using the lateral earthquake design force, V or V_d, of

NBC Article 4.1.8.11. or 4.1.8.12. be multiplied by R_dR_o/I_E to obtain realistic values of anticipated maximum deflections. If the overall building structure is made up of types of SFRSs having different values of R_dR_o, then the value of R_dR_o used for the determination of the lateral earthquake design force must be used. The earthquake importance factor, I_E, is used to increase the design loads and to reduce the inelastic demand on the structure for the DGM; it is not meant to amplify the DGM, as the resulting return period would be much larger than 2 475 years. The importance factor reduces the product R_dR_o so that realistic values of anticipated deflections are obtained when the results of a linear analysis are multiplied by R_dR_o/I_E. In all cases, the effects of torsion, including those due to accidental eccentricities, are to be included in the calculation of lateral deflections. With the inclusion of torsional effects, the largest deflection is at one of the two extreme edges of the building and not at the centre of mass. Increases in displacement and drift caused by foundation movements (primarily rotations) and GILD (Type 9 irregularity) must also be accounted for.

NBC Sentence 4.1.8.13.(3)

196. The deflection parameter that best represents the potential for structural and non-structural damage is interstorey deflection, also known as interstorey drift. Lateral deflection at the top of a structure is not a good indicator of damage potential because the various types of SFRSs have different deflection profiles along their heights. NBC Sentence 4.1.8.13.(3) specifies a limit, known as a drift limit, on the largest interstorey deflection at any level of the structure. Ordinarily, the drift limit is $0.025h_s$, where h_s is the interstorey height, but for post-disaster buildings and High Importance Category buildings, the drift limits are $0.01h_s$ and $0.02h_s$, respectively. An interstorey deflection of $0.025h_s$ defines a state of extensive damage in a building; larger interstorey deflections are in the realm of severe damage and are to be avoided. Simply complying with the drift limit may help contain architectural damage to an extensive level; however, it should be noted that extensive structural damage can occur at interstorey deflections well below the drift limit. The potential for structural damage is determined by checking the realistic displacements of the structure, the ductility demands in the non-linear elements of the SFRS, and the structural detailing requirements. The rest of the structure is also checked and modified to ensure that it is able to carry the gravity loads at the realistic displacements, which may involve changing the detailing in the members or reducing the displacements by stiffening the structure.

197. The more stringent drift limit of $0.01h_s$ for post-disaster buildings reflects the need for facilities such as hospitals, power generation stations and fire stations to remain operational following an earthquake. A report by the SEAOC Vision 2000 Committee[99] specifies an operational performance level drift limit of $0.005h_s$ for such buildings, but this limit is associated with ground motions at a probability of exceedance of 10% in 50 years. Because the DGMs are specified at a probability of exceedance of 2% in 50 years in the NBC, the drift limit of $0.01h_s$ in NBC Sentence 4.1.8.13.(3) is consistent with the operational performance level defined by the SEAOC Vision 2000 Committee.[99]

Structural Separation (NBC Article 4.1.8.14.)

NBC Sentence 4.1.8.14.(1)

198. The provisions in NBC Subsection 4.1.8. are based on the assumption that the building being designed is a stand-alone building that will not interact with any other building during its response to an earthquake. Observations of building behaviour during actual earthquakes have demonstrated that collisions between buildings can lead to extensive damage, particularly if adjacent buildings have different heights and floor spacings. Filiatrault et al.[100] discuss the effects of the pounding of buildings during earthquakes. To avoid pounding, there must be a separation between the building being designed and any adjacent building that is adequate for both to undergo seismically induced deflections without any contact. NBC Sentence 4.1.8.14.(1) requires that the minimum separation be equal to the square root of the sum of the squares of the calculated deflections of the two buildings, as recommended by Filiatrault et al.[100] and by Filiatrault and Cervantes.[101] Adjacent buildings are likely to vibrate out of phase at different periods rather than in phase at the same period.

199. The deflections for the adjacent existing building must be calculated on the same basis as those for the building being designed.

200. If it is not feasible to separate the two buildings by sufficient distance, then the buildings must be connected to each other. The requirements for connections are given in NBC Sentences 4.1.8.14.(2)

to (4); they also apply to expansion joints within buildings, which must be designed for the appropriate seismic forces or detailed to ensure that earthquake damage is confined to the joint and does not affect the principal structural elements.

NBC Sentence 4.1.8.14.(2)

201. When two buildings are connected, their response to earthquake ground motions will be interactive, i.e., they will respond as a single structural system rather than as two independent systems. Although the NBC does not specify how such an interactive system should be analyzed, it does require that the method of connection take into account the properties of each building (i.e., mass, stiffness, strength and ductility), the properties of the connections, and the anticipated response of the connected buildings. The displacement compatibility of the two buildings should also be considered. The modeling of the connections and of the elements of each building must meet the requirements of NBC Sentence 4.1.8.3.(8). If the adjacent building is an existing structure, then the modeling of its elements must be based on its as-built characteristics. In addition to using the results of the analysis for the design of the connections between the buildings, it is advisable to review the capability of the existing structure to perform adequately once connected to the new building. In conducting such a review, the performance standards for the existing building, e.g., interstorey drift limits, should be based on the requirements of the NBC 2015 rather than on the edition of the NBC in effect when the existing building was constructed.

NBC Sentence 4.1.8.14.(3)

202. Where buildings are rigidly connected, the loading and design must be based on the lowest value of R_dR_o of the individual buildings. This requirement ensures that the loading and design requirements for the component building with the lowest ductility and overstrength will govern, and is based on the assumption that the performance capacity of the connected buildings will be limited by the capacity of the building with the lowest value of R_dR_o. Buildings can be considered to be rigidly connected if the connection enables both buildings to undergo the same lateral deflection at each storey.

NBC Sentence 4.1.8.14.(4)

203. As previously noted, buildings interconnected with non-rigid elements or energy-dissipating elements, such as friction or viscoelastic dampers, will behave as an interactive structural system. Due to the complexity of such a system—particularly if one component is an existing building—this Sentence requires that a special study be carried out instead of simply applying the loading and design requirements of NBC Subsection 4.1.8.

Design Provisions (NBC Article 4.1.8.15.)

204. This Article specifies a number of design requirements that are essential for an SFRS and its elements to perform satisfactorily during strong earthquake ground shaking. One of the objectives of seismic design is to prevent structural collapse by ensuring that inelastic behaviour is confined to those elements that can dissipate energy inelastically during reversing cycles of deformation without loss of capacity. One of the important ways of achieving this goal is to ensure that elements with poor energy dissipation characteristics are designed with sufficient strength so that they will not yield. This approach is one of the key features of what is known as the capacity design philosophy (Paulay and Priestley[46]). The New Zealand standard NZS 4203:1992[72] requires that capacity design be used for all ductile structures. Although the NBC 2015 does not require that the capacity design philosophy be followed in the design of all structures, the use of capacity design principles in the design of ductile structures is prescribed in the CSA material standards for concrete (CSA A23.3), steel (CSA S16), wood (CSA O86) and masonry (CSA S304, "Design of Masonry Structures"); in addition, NBC Article 4.1.8.15. includes several specific provisions that are based on the capacity design philosophy.

NBC Sentence 4.1.8.15.(1)

205. The primary purpose of diaphragms is to transfer lateral loads from their origin (i.e., inertial forces throughout the building) to the elements that resist those loads (e.g., walls or frames). Typically, diaphragms consist of some combination of slabs, steel deck, deep beams and trusses. Although such elements are subject to axial, shear and bending actions, their primary action in resisting earthquake loads as part of the SFRS is in shear. Since most of these types of elements have very poor energy dissipation characteristics in shear, they should be designed so as not to yield, except as permitted by NBC Sentences 4.1.8.15.(2) and (3). An important aspect of avoiding yielding is to ensure that the components of the diaphragm are well tied together so that they act as a unit. Since stress concentrations are likely to occur near openings in a diaphragm, the design of the diaphragm must account for any openings. Also, since the connections between diaphragms and SFRS elements (e.g., wall anchorages) are extremely important to maintaining the integrity of the structure, they should also be designed not to yield.

Clause (a): In order to ensure that the diaphragm does not yield, it must be designed so that the forces applied to it reflect the strengths of the SFRS and of the elements of the SFRS to which the diaphragm is connected, rather than just the calculated lateral earthquake loads. The forces applied to the diaphragm arising from the lateral earthquake loads (as determined from NBC Article 4.1.8.11. or 4.1.8.12.) must be increased to reflect the actual strength of the SFRS when subjected to lateral loads. For example, if the actual base shear capacity of the SFRS is 20% larger than the design base shear, V, then the shear forces applied to the diaphragm due to the lateral earthquake loads must also be increased by 20%. In addition, forces must be applied to the diaphragm to account for the transfer of loads between lateral-load-carrying elements of the SFRS; such load transfers can result from offset walls, where the offset is in- or out-of-plane, and discontinuous walls resting on columns, where the discontinuities generate large in-plane diaphragm forces. The diaphragm design forces must be associated with the actual capacities of the lateral-load-carrying elements of the SFRS and must also account for discontinuities and changes in stiffness in these elements. By designing for these capacity-based loads, yielding in the diaphragm will be prevented because connecting elements will of necessity yield first and the amount of load that they can transfer is limited by their capacities. CSA A23.3 contains detailed design requirements for structural diaphragms subject to earthquake-induced forces.

Clause (b): Regardless of the value of the diaphragm design force calculated in accordance with Clause (a), the diaphragm at any level x must be designed for a minimum shear force corresponding to the design base shear, V, divided by the total number of storeys, N. This minimum shear force, which represents the average shear per storey, ensures that there is adequate protection for diaphragms in the lower part of the building, for which the diaphragm shear calculated from the load distribution in NBC Sentence 4.1.8.11.(7) is quite low.

NBC Sentences 4.1.8.15.(2) and (3)

206. Where diaphragms and their connections are designed to remain elastic, the design forces applied to the diaphragms must meet the capacity design requirements of NBC Sentence 4.1.8.15.(1). Alternatively, where steel deck roof diaphragms in buildings of less than 4 storeys and wood diaphragms in roofs and floors are designed and detailed in accordance with the applicable CSA standard (i.e., CSA S16 or CSA O86) to exhibit ductility and energy dissipation characteristics, the design forces applied to the diaphragms are permitted to be reduced. The ability of steel deck roof diaphragms to exhibit a ductile inelastic response is determined from reversed cyclic loading tests (Essa et al.[102] and Tremblay et al.[103]) and from Non-linear Dynamic Analyses (Tremblay and Rogers[104]). However, the ductility of steel deck roof diaphragms is generally limited, and excessive inelastic deformations may concentrate at the edges of the diaphragm near the vertical elements of the SFRS under dynamic loading (Cohen et al.[105] and Massarelli et al.[106]), as shown in Figure J-26. A similar response under dynamic loading is expected at discontinuities in the diaphragm and at locations of reduced shear resistance along the diaphragm span, L. Until additional research data and/or diaphragm systems with enhanced ductile behaviour become available, the inelastic response in steel deck roof diaphragms should be assumed to be limited and should not be considered as the main energy-dissipating system of a building.

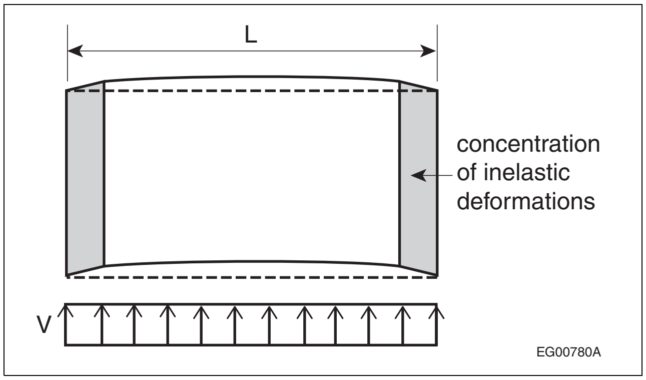


Figure J-26
Concentration of inelastic deformations at the edges of a diaphragm near the vertical SFRS elements

NBC Sentence 4.1.8.15.(4)

207. The in-plane flexibility of roof and floor diaphragms magnifies the dynamic response of buildings to earthquakes. This dynamic magnification is particularly significant in single-storey buildings with flexible roof diaphragms made of wood or untopped steel deck panels and is considered in the calculation of their fundamental lateral period (see NBC Sentence 4.1.8.11.(4)). The flexibility of such roof diaphragms magnifies the ductility demand on the vertical elements of the SFRS (Tremblay and Stiemer,[73] Adebar et al.,[68] and Humar and Popovski[79]) and increases the in-plane force and deflection demands on the roof diaphragm (Tremblay and Stiemer,[73] Medhekar and Kennedy,[107] Tremblay et al.,[75] and Massarelli et al.[106]). Because these effects increase the level of inelastic response in the SFRS, Non-linear Dynamic Analysis must be used to assess them. These effects are generally more pronounced for higher levels of diaphragm flexibility, which is characterized by the ratio Δ_D/Δ_B, where Δ_D is the maximum elastic in-plane deflection of the diaphragm relative to the average deflection of the vertical SFRS elements adjoining the diaphragm, and Δ_B is the average storey drift of the two adjoining vertical SFRS elements due to a horizontal load uniformly distributed along the diaphragm span. The magnification of the inelastic response due to the effects of diaphragm flexibility needs to be considered where the ratio Δ_D/Δ_B exceeds 0.5 and the value of R_d of the SFRS used in design is greater than 1.5.

The magnification of the ductility demand on the vertical elements of the SFRS can be addressed by comparing the anticipated ductility demand with the inelastic deformation capacity of the elements, or by increasing the strength of the elements so that the ductility demand is consistent with the value of R_d specified for the SFRS.

In the first approach, the expected seismic deformations of the vertical SFRS elements must be properly assessed, and the performance of these elements under reversed cyclic inelastic loading producing such deformations must be demonstrated to be satisfactory. The deformation demand on the vertical SFRS elements can be estimated by using the simplified expression given in NBC Subclause 4.1.8.15.(4)(a)(i), rather than by performing a detailed Non-linear Dynamic Analysis. This simplified expression assumes that the maximum lateral displacement of the building, including the inelastic response of the vertical SFRS elements and the effects of diaphragm flexibility, is equal to the total elastic displacement, $R_oR_d(\Delta_B + \Delta_D)$, and that the in-plane deflection of the diaphragm corresponds to that calculated under a static lateral load of R_oV (see Figure J-27). The ductility demand, μ_B, imposed on the vertical SFRS elements is obtained by subtracting $R_o\Delta_D$ from the total elastic displacement and dividing the result by $R_o\Delta_B$. For buildings with a very flexible roof diaphragm, it is recommended that the total elastic displacement be assessed using the value of the fundamental lateral period obtained from an established method of mechanics in accordance with NBC Clause 4.1.8.11.(4)(c). As suggested by Adebar et al.,[68] where the resistance of the vertical elements of the SFRS is known, the diaphragm deflection can be calculated under a lateral load corresponding to this resistance instead of under a lateral load of R_oV.

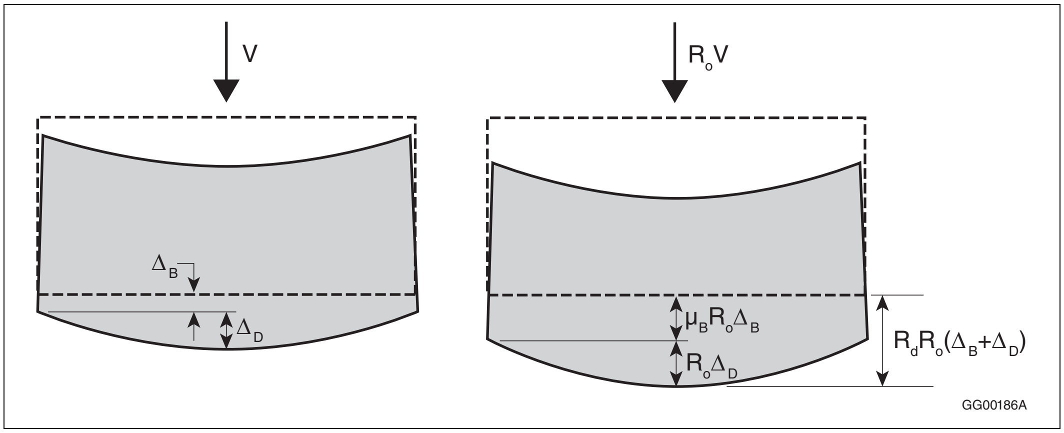


Figure J-27
Plan view of the lateral deformation of a flexible roof diaphragm under the lateral earthquake design force, V, and at the maximum lateral displacement of the building under a lateral load of R_oV

Inelastic deformation capacities are given in CSA S16 for vertical SFRS elements consisting of steel moment-resisting frames of Types D (ductile), MD (moderately ductile) and LD (limited ductility) and eccentrically braced frames of Type D. CSA S16 requires seismic qualification testing for the bracing members of steel buckling-restrained braced frames of Type D. The test protocol can be used as given or can be adjusted to reflect the anticipated deformation demand. The test results can be used to verify the adequacy of the SFRS. Test data on the deformation capacities of certain other types of vertical SFRS elements is available; for instance, testing has indicated that concentrically braced steel frames of Type MD can withstand storey drifts larger than 1.5% without failure (Lumpkin et al.[108] and Palmer et al.[109]).

For concrete tilt-up construction where the vertical SFRS consists of moderately ductile (R_d = 2.0) walls and frames, CSA A23.3 requires a displacement-based design approach that explicitly accounts for the inelastic displacement demands on wall panels. Solid wall panels in tilt-up construction dissipate energy through yielding of the panel-to-panel and panel-to-base connections. The displacement capacities of standard tilt-up connectors have been investigated by Lemieux et al.[65] and Devine et al.[66] Solid wall panels can be designed to rock individually or as groups. Wall panels with large openings, i.e., frame panels, in tilt-up construction dissipate energy primarily through yielding of the legs (columns) at the base of the panel. On the basis of the experimental work of Dew et al.,[67] CSA A23.3 requires tilt-up frame panels to satisfy the requirements for moderately ductile cast-in-place moment-resisting frames where the inelastic rotational demand on any member in the tilt-up frame panel exceeds 0.02 radians, in which case the inelastic rotational capacity is 0.04 radians.

The second approach for addressing the magnification of the ductility demand on the vertical elements of the SFRS is adopted in situations where the deformation capacity of the elements is exceeded, and involves increasing the strength of the elements to limit the ductility demand on them. Humar and Popovski[79] have proposed adjustment factors to be applied to the seismic design loads on vertical SFRS elements to ensure that the ductility demand on the elements does not exceed a target value. Similar seismic design forces are obtained when the magnification factor specified in NBC Subclause 4.1.8.15.(4)(a)(ii) is applied. Diaphragm flexibility can significantly affect the level of strength required to resist the seismic design forces. For instance, for a diaphragm with Δ_D/Δ_B = 2.0 in a structure designed with R_d = 4.0, the seismic design forces for the SFRS must be magnified by the following factor:

$$\mathrm{R_d}\frac{\left(1+\frac{\Delta_D}{\Delta_B}\right)}{\left(\mathrm{R_d}+\frac{\Delta_D}{\Delta_B}\right)} = 4.0\frac{(1+2.0)}{(4.0+2.0)} = 2.0$$

However, the strength required to resist the magnified forces need not exceed the value corresponding to R_d = 1.5.

The forces and deflections in flexible roof diaphragms are magnified because of the increased contribution of higher modes of vibration to diaphragm deformations; the higher modes do not benefit from the damping effect of energy dissipation in the vertical SFRS elements. This situation is analogous to the increase in shears and moments that is induced by higher modes in shear walls yielding in flexure at their bases. In particular, the distribution of shears along the diaphragm span can deviate significantly from the linear variation predicted by static analysis. In the example illustrated in Figure J-28, Non-linear Dynamic Analysis shows that shears with a magnitude corresponding to the factored shear resistance, S_r, of the vertical SFRS elements occur up to a distance of about 30% of the diaphragm span from the diaphragm edges; this demand is not predicted by the ESFP or the Modal Response Spectrum Method (Trudel-Languedoc et al.[81]). A more detailed discussion of this behaviour, which requires further study, can be found in Mortazavi and Humar.[110]

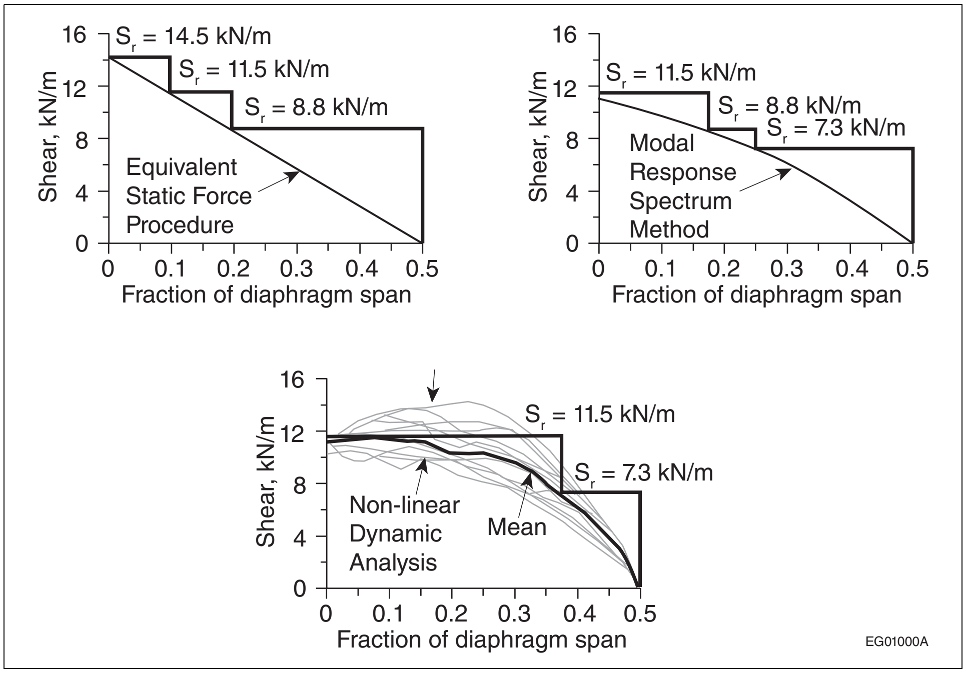


Figure J-28

Distribution of shears along half the span of a diaphragm, determined using the ESFP, the Modal Response Spectrum Method and Non-linear Dynamic Analysis (adapted from Trudel-Languedoc et al.[81])

NBC Sentence 4.1.8.15.(5)

208. An SFRS that includes an in-plane discontinuity in a vertical lateral-force-resisting element is designated as having a Type 4 irregularity (see NBC Table 4.1.8.6.). Where $I_E F_a S_a(0.2) \geq 0.35$, NBC Sentence 4.1.8.6.(3) specifies that the requirements for irregular structures listed in NBC Table 4.1.8.6. must be satisfied. For a Type 4 irregularity, NBC Sentence 4.1.8.15.(5) requires the use of a capacity design approach in which the elements supporting any discontinuous wall, column or braced frame are designed for the forces transferred from above the discontinuity that are associated with the lateral load capacity of the structure. The design of these supporting elements must be based on the actual capacity of the discontinuous elements they support rather than only on the forces generated by the loads specified in NBC Article 4.1.8.11. or 4.1.8.12. Using the capacity as the basis for the design of these supporting elements ensures that yielding will not occur at or below the discontinuity.

NBC Sentence 4.1.8.15.(6)

209. NBC Sentence 4.1.8.9.(4) specifies the value of $R_d R_o$ to be used where there is a vertical variation of $R_d R_o$, i.e., where the type of SFRS changes at one or more levels in the structure. NBC Sentence 4.1.8.15.(6) specifies a capacity design approach in which the elements of the SFRS below the level where the change in $R_d R_o$ occurs are designed for the forces associated with the lateral load capacity of the SFRS above that level. Typically, the upper portion of the SFRS is more ductile and therefore has less strength than the lower portion. In such cases—where the upper portion is designed for higher values of R_dR_o than the lower portion—the forces in the upper portion are typically lower than they would be if the same value of R_dR_o were used throughout the building. NBC Sentence 4.1.8.15.(6) requires that the design forces in the lower portion be not less than the capacity of the upper portion so as to avoid a weak lower level and the undesirable concentration of all the yielding in this less ductile level.

NBC Sentence 4.1.8.15.(7)

210. NBC Article 4.1.8.8. requires that earthquake forces be assumed to act in any horizontal direction; this requirement can be met by independent analysis and design along two orthogonal horizontal directions. In many cases, some of the elements of the SFRS will be subject to forces from two loading directions (e.g., columns common to two orthogonal moment frames or orthogonal walls that are part of a central core). Because the design loads are reduced to take inelastic response into account, simultaneous yielding in both directions is likely to occur. In accordance with the capacity design philosophy, NBC Sentence 4.1.8.15.(7) requires that, in such situations, account be taken of the potential for concurrent yielding of other elements framing into the column or wall from all directions, both at the level under consideration and as appropriate at other levels.

NBC Sentence 4.1.8.15.(8)

211. Capacity design principles have been adopted in the NBC for seismic design in Canada. However, the SFRS includes both ductile components that are specifically designed and detailed to withstand cyclic inelastic deformations and non-yielding components that are designed to remain essentially elastic. The ductile components are sized for the effects of gravity loads combined with earthquake loads reduced by R_dR_o. The capacity-protected non-yielding components are designed to carry the gravity loads and to resist the seismic force effects generated upon yielding of the ductile components. In cases where a ductile component of the SFRS is overly strong—which may be the case when its design is governed by drift limits, minimum reinforcement requirements or gravity loads—the design forces for the affected capacity-protected non-yielding elements may become very large; the NBC therefore prescribes an upper limit on such forces, which is equal to the seismic force determined with $R_dR_o = 1.0$, that is, the elastic force level. In recognition of the fact that the non-yielding elements may possess some overstrength and ductility, the upper limit on the design forces can be reduced, where permitted in the applicable CSA material standard, to a level lower than the elastic level, but the reduced upper limit must not be less than the seismic force determined using $R_dR_o = 1.3$.

Sample Determination of the Upper Limit on the Design Forces Associated with the Lateral Capacity of the SFRS

212. The analysis of a building indicates that the factored bending moment and the factored shear force applied to a wall are 3 000 kN·m and 300 kN, respectively. These values were calculated using the appropriate R_dR_o for the SFRS. Capacity design requires that the wall yield (i.e., dissipate inelastic energy) in flexure before failing in shear. Thus, the factored shear resistance must be greater than the shear associated with the nominal or probable flexural resistance of the wall, in accordance with the applicable CSA material standard. Because the wall has a very large flexural overstrength, its actual flexural resistance is 10 000 kN·m. According to the moment-to-shear ratio obtained from the building analysis, the shear associated with this bending moment is 1 000 kN. If the building is analyzed to determine the elastic force demands using $R_dR_o = 1.0$, the shear force applied to the wall is 900 kN, and if it is analyzed using $R_dR_o = 1.3$, the shear force reduces to 900/1.3 = 692 kN. Thus, a strict application of the capacity design requirement would require that the wall have a factored shear resistance of 1 000 kN, whereas the upper limit on the design forces corresponding to the elastic force ($R_dR_o = 1.0$) level requires that the wall have a factored shear resistance of 900 kN. The reduced upper limit corresponding to $R_dR_o = 1.3$, where permitted by the applicable CSA material standard, requires that the wall have a factored shear resistance of 692 kN. Note that if the wall had an actual flexural resistance of 5 000 kN·m, the shear force associated with this bending moment would be 500 kN according to the moment-to-shear ratio from the building analysis. In this case, the wall would have to be designed for a shear force of 500 kN, and the upper limit on the design forces of 900 kN or 692 kN noted above would not be relevant.

NBC Sentence 4.1.8.15.(9)

213. This Sentence provides an exception to the capacity design philosophy of protecting the foundation, by allowing its factored overturning resistance to be less than the lateral load overturning capacity of the SFRS that it supports, provided the conditions stated in NBC Sentence 4.1.8.16.(4) are met. NBC Sentence 4.1.8.15.(9) specifies, however, that the design and the values of R_d and R_o for the type of SFRS used must nevertheless conform to NBC Table 4.1.8.9. Accordingly, less ductile foundation designs cannot be used for SFRSs with a large overstrength (i.e., with a smaller value of R_dR_o) unless they are allowed by NBC Table 4.1.8.9. Previous editions of the NBC imposed an upper limit on design forces for rocking foundations; this upper limit no longer applies in the NBC 2015, as recent studies have shown that, for many soil types, it is unconservative (Adebar et al[111]).

NBC Sentence 4.1.8.15.(10)

214. NBC Sentence 4.1.8.15.(10) introduced in the NBC 2015 specifies that foundation displacements and rotations must be considered in the design, as required by NBC Sentence 4.1.8.16.(1).

Foundation Provisions (NBC Article 4.1.8.16.)

NBC Sentence 4.1.8.16.(1)

215. A common assumption in the analysis of how structures respond to earthquakes is that foundations are rigid and do not displace under earthquake loading. However, foundations supporting the SFRS will displace under earthquake loading and these foundation displacements will change the deflections and drifts of the structure. This can affect the force distribution in the SFRS and increase the displacement demand on the rest of the structure. Failure of the gravity-load-carrying system is a common threat to life safety in an earthquake, so it is important that any increase in displacements and drifts of the gravity-load-resisting system be considered.

216. For some types of foundations, the effects due to displacements will be small: examples among types of frame columns are piled foundations, raft foundations, and spread footings with soil or rock anchors; examples among types of walls and braced frames are large piled foundations, raft foundations and large spread footings with soil or rock anchors. These types of foundations are typically restrained in some fashion, by piles or soil anchors for example, and behave more or less elastically on the soil or rock. However, the behaviour of large unrestrained spread footings under a shear wall or a braced frame resisting overturning moments is more complicated and varies. The examples of rotating foundation behaviour that follow are based on the typical factored load design, i.e., ultimate limit state (ULS) design, where the factored soil or rock resistance is about 50% of the ultimate resistance (a resistance factor of about 0.5):

(a) Footings where the resultant of the bearing stress in the soil or rock due to the applied vertical load and overturning moment is within the middle third of the footing and where there is no uplift of the heel of the footing will undergo relatively small rotations. The overturning moment–foundation rotation response will be approximately within the initial linear range, as long as the bearing stress is low enough to prevent significant deformation of the soil at the toe of the footing.

(b) Footings where the resultant of the bearing stress in the soil or rock is within the footing but the heel of the footing lifts up (see Figure J-29) can undergo quite large rotations. The footing is typically designed using a uniform stress block in the soil or rock that is based on the factored resistance of the soil or rock. This case is quite non-linear because of the uplift of the footing and there will likely be some additional non-linearity in the soil.

(c) Footings where the applied overturning moment—assumed to be equal to the overturning capacity of the SFRS—is such that the required eccentricity of the soil bearing stress (see Figure J-29) is larger than is possible given the length of footing and the factored bearing resistance of the soil or rock. In this case, the footing is not as strong as the capacity of the SFRS and the system is not in static equilibrium. However, under certain conditions, it can be in dynamic equilibrium: commonly referred to as a “rocking footing,” this case is highly non-linear due to the uplift of the footing, the non-linear behaviour in the soil, and the rocking of the footing. As a result, the footing can undergo very large rotations.

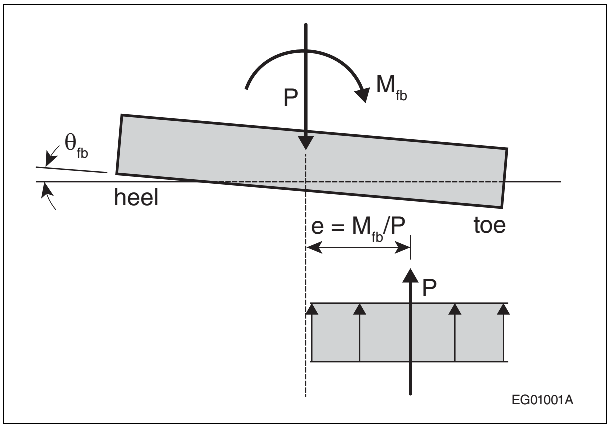


Figure J-29

Static equilibrium of a rotating footing with uplift at the heel and significant soil strains at the toe of the footing due to an applied overturning moment, M_{fb}

Note to Figure J-29:

(1) The resultant of the uniform bearing stress in the soil or rock is equal to the applied vertical load, P, which includes the weight of the footing.

217. Generally, the relationship between overturning moment and foundation rotation is non-linear (see Figure J-30).

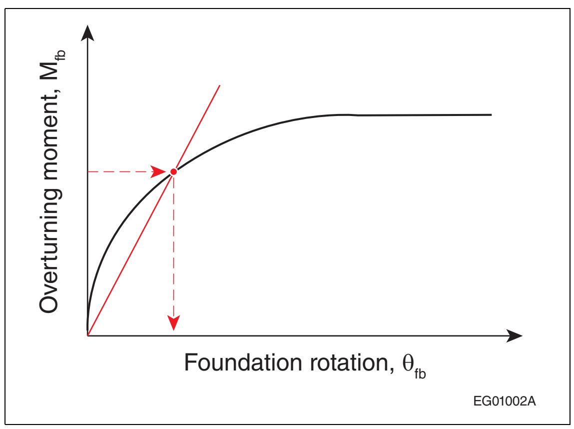


Figure J-30

Typical overturning moment–foundation rotation relationship for a spread footing that is not restrained against rotation

When the foundation overturning moment–rotation response is in the initial linear range, such as in case (a) of Paragraph 216, the rotation of the footing can be calculated using equations for the linear rotational stiffness of footings, such as those given in ASCE/SEI 41, or using standard structural analysis techniques. If the footing is in the non-linear range, using an effective secant stiffness (lower than the initial stiffness) of the foundation in a structural analysis may be inappropriate as it will result in a more flexible structure, and hence underestimate the design forces and overestimate the displacements. A simple approach suggested in CSA A23.3 is to assume that the foundation is rigid when calculating the design forces, and then calculate the displacements of the structure by adding the movements of the SFRS due to the foundation rotation to the calculated deformation of the SFRS. CSA A23.3 also provides a simplified method for determining the rotation of a foundation that has a greater capacity than the SFRS (cases (a) and (b) of Paragraph 216), referred to as a "capacity-protected" foundation. Additionally, CSA A23.3 includes simple upper-bound estimates for the additional movements of SFRSs supported on foundations that are not in static equilibrium (i.e., not capacity-protected, as in case (c) of Paragraph 216) but that satisfy the requirements of the NBC.

As previously stated, for many types of structures, the foundation rotations will be very small and, in the designer's judgement, the increased displacements of the SFRS may be of little consequence. However, the effects may be significant in certain cases; for example:

(a) A tall concrete or masonry shear wall extending through several levels of below-grade concrete parking slabs that are attached to foundation walls; the footing is not tied down and sits on soft soil: In this scenario, the footing rotation might be small but it might have a significant effect on the shear distribution in the below-grade portion of the shear wall. The effect can be bounded by treating the wall as fixed and then as pinned at the footing. Alternatively, a rotational spring can be developed for the footing and used instead of a pinned base in the analysis. This scenario presents a complex problem, even if the base is considered fixed and involves wall and slab flexural and shear stiffnesses, both cracked and uncracked, as well as soil stiffness. Considerable engineering judgement is needed.

(b) A footing at grade that is attached to an SFRS where both the footing and the SFRS are free to rotate; the footing is not on piles or on a raft foundation, nor is it tied down, and the SFRS is not attached to and constrained by surrounding walls at the lower level: In this scenario, the footing rotation manifests itself as an additional more or less constant drift up the height of the building, potentially in the order of 0.5% to 1.0%, which may be a concern for the gravity-load-resisting system. Of particular concern would be cases where heavily loaded columns are fixed to a stiff transfer girder near the ground level where drifts are typically considered to be very small.

NBC Sentence 4.1.8.16.(2)

218. There are several important reasons why foundations need to be designed so as to prevent damage during the design earthquake ground motion, two being that foundation damage is hard to identify and difficult to repair. Of particular significance is the fact that the foundation is the mechanism for transmitting earthquake loads to the structure; damage to the foundation would place the building at risk even if the structure itself were not damaged. In order to minimize the likelihood of foundation damage, NBC Sentence 4.1.8.16.(2) requires that foundations be designed with sufficient factored shear resistance to resist the lateral load capacity of the SFRS, regardless of the earthquake loads used to design the SFRS. This is a particularly important application of the capacity design philosophy, as it allows the structure to dissipate energy inelastically while the foundation remains essentially linearly elastic. Exceptions to the requirement for designing foundations for the overturning capacity of the SFRS are given in NBC Sentence 4.1.8.16.(3), which allows an earthquade design force cut-off, and in NBC Sentence 4.1.8.16.(4), which allows the footing resistance to be less than the SFRS capacity demand. Note that the lateral load capacity of the SFRS is the actual capacity based on the final design and it can be affected by gravity load design, wind design, minimum detailing requirements, strain hardening, actual yield strength, etc. The increase over the factored earthquake load, in many cases, is much larger than the R_o value. See CSA A23.3 for further guidance on the design of structures with concrete foundations.

NBC Sentence 4.1.8.16.(3)

219. NBC Sentence 4.1.8.15.(8) allows the earthquake design force to be calculated at $R_dR_o = 1.0$, unless the material standards allow the value to be based on $R_dR_o = 1.3$, which recognizes the inherent overstrength in most structures. However, gravity load provides, or contributes to, the overturning resistance of many elements: e.g., resistance to uplift in frame columns and footings, and overturning resistance of foundations in general. There is no inherent overstrength component in the gravity load so a force cut-off based on $R_dR_o = 1.3$ is not appropriate; a force cut-off based on $R_dR_o = 1.0$ is more appropriate for gravity load elements. To capture overstrength of the soil or rock, a factor of 1.5 is applied to the bearing resistance as the typical ultimate resistance of the soil or rock is about 2.0 times the factored resistance. For sliding resistance and other elements in the foundation, such as piles, caissons, soil or rock anchors, multiplying the factored resistance by 1.3 is appropriate. The increase in the overturning capacity of a foundation due to the application of a 1.5 factor on the bearing resistance of the soil or rock will depend on the length of the calculated uniform bearing stress. There may be significant foundation overstrength on weaker soils where the increase in bearing stress results in a significant shifting of the vertical force resultant in the soil outwards towards the toe of the footing. On the other hand, there may be very little overstrength available for footings on very hard soil or rock because the vertical force resultant in the soil or rock due to the factored bearing stress is already at the toe of the footing.

NBC Sentence 4.1.8.16.(4)

220. The concept of rocking footings was introduced in the 1994 edition of CSA A23.3 and in the 2005 edition of the NBC. The requirements were based on New Zealand codes as well as non-linear studies focusing on displacements of the SFRS. New studies on the topic, completed in 2014 (Adebar et al.[111]), examined displacements and focused more on drift ratios because of their importance to the gravity load frame. The studies indicated that:

(a) structures on soft soils do not behave the same as the classic rigid block rocking on a rigid surface;
(b) soil overstrength capacities allow some foundations on soft soils to resist increases in lateral loads, so the notion that rocking limits the lateral load does not always hold;
(c) footing rotations can produce drifts that can affect the design, even when the footing is stronger than the wall;
(d) when the wall is stronger than the footing, the footing rotations can become significant and produce large soil stresses; and
(e) the ratio of wall capacity to footing capacity is a better parameter to predict behaviour than simply having an R_dR_o force cut-off.

To apply NBC Sentence 4.1.8.16.(4), the footing and supported SFRS must be free to rotate and uplift on the soil or rock. Many typical structures are constrained in some way and so do not satisfy this condition. Examples are:

(a) footings on piles, caissons, or drilled piers;
(b) footings with soil or rock anchors to resist uplift;
(c) raft foundations;
(d) walls that extend through below-grade structures where the floor diaphragms are attached to foundation walls.

All constrained structures must be designed with a foundation overturning resistance that is greater than the overturning capacity of the supported SFRS.

NBC Sentence 4.1.8.16.(5)

221. This Sentence specifies that the capacities of the soil and rock on which the foundation rests are not to be exceeded during the DGM. The evaluation of those capacities must take into account the potential for degradation due to large reversing strains. Also, it is required that the foundation not undergo large lateral displacements during an earthquake due to the loss of strength of the soil.

NBC Sentence 4.1.8.16.(6)

222. In addition to the requirements specified in NBC Sentences 4.1.8.16.(2) and (5), for the foundation to perform satisfactorily, it must both act as an integral unit and provide a continuous load path from the structure into the ground. Achieving this requires special attention when the foundation is made up of independent elements such as piles, drilled piers or caissons. NBC Sentence 4.1.8.16.(6) imposes specific requirements regarding the integral functioning of foundation elements other than in cases of low DGMs, i.e., $I_EF_aS_a(0.2) < 0.35$.

Clause (a): To prevent columns or walls from moving relative to each other, piles or pile caps, drilled piers and caissons need to be interconnected in at least two directions by continuous ties, which may consist of grade beams or slabs, or a combination of both. Such foundation elements are often used in soft or loose soils, which do not have the capacity to provide lateral restraint near the ground surface. Ties are required to provide lateral restraint both to prevent damage to the structural elements immediately above the foundation and to prevent the spreading of and subsequent damage to piles, drilled piers and caissons. Design force requirements for such ties are specified in NBC Sentence 4.1.8.16.(9).

Clause (b): In addition to the foundation elements needing to be tied together as per the requirements in Clause (a), it is necessary to prevent displacements due to sliding between these elements and the building structure. Clause (b) requires that they be embedded at least 100 mm into the structure or the pile cap (which is integral with the structure), a depth deemed to be sufficient to provide lateral continuity between the structure and the foundation units.

Clause (c): As the structure displaces laterally, the overturning moment generated at its base by the design seismic load may result in a net tension in the piles, drilled piers or caissons at or near the outer edges of the foundation; in accordance with NBC Sentence 4.1.8.16.(5), the connections from the structure to the foundation elements must be designed for such tension in order to prevent separation between the structure and the foundation elements. When the effects of the design seismic load do not result in any tension (owing to the counteracting effects of gravity loads), Clause (c) requires that the connections between the structure and the foundation elements be designed for a minimum tension force equal to 15% of their factored compression capacity. This relatively small nominal tension capacity in the connection is deemed to be necessary to prevent separation in the event the overturning effects should accidentally result in a small amount of tension, and to provide integrity at the joint to assist in transferring shear between the pile and the cap. Connections to wood piles are exempted from this requirement because the low lateral pile capacity should be able to be transferred by the minimum embedment.

NBC Sentence 4.1.8.16.(7)

223. In regions of moderate to high seismicity, i.e., where $I_EF_aS_a(0.2) \geq 0.35$, basement walls must be designed to resist increased lateral pressure due to the movement of backfill or natural ground associated with earthquake ground motions (see Mononobe and Matsuo,[112] and Seed and Whitman[113]). Such basement walls are normally considered “non-yielding” in that the restraints at the top and bottom of these walls prevent the small amount of movement required to develop minimum active earth pressures. FEMA 369 provides information on the dynamic forces acting on a non-yielding wall on a rigid base.

There are two bounding cases to consider: those in which the basement wall will deform enough to develop active pressure and those in which it is considered effectively rigid. Deforming basement walls are usually designed on the basis of the Mononobe–Okabe theory (Mononobe and Matsuo,[112] and Okabe[114]) and adaptations of this theory, such as those proposed by Seed and Whitman.[113] The assumption of active pressure is based on the displacements of the backfill during construction, the flexibility of the basement walls between floors, and the stress-strain behaviour of the backfill. Seismic pressures against effectively rigid walls can be evaluated using Wood’s[115] equation. In both of these bounding cases, the seismic action is generally specified by an inertial force that is based on the full PGA. However, significant evidence has been provided by centrifuge tests (e.g., Sitar et al.,[116] Geraili Mikola,[117] Al Atik,[118] Al Atik and Sitar,[119] Lew et al.,[120] and Lew[121]) and numerous non-linear analyses (e.g., Amirzehni,[122] and Amirzehni et al.[123]) to justify using a fraction of the PGA in assessing the inertial force. This conclusion is supported by the fact that there are no known cases of basement wall failure due to seismic loading.

NBC Sentence 4.1.8.16.(8)

224. Additional design requirements for foundations located in regions of high seismicity, i.e., where $I_EF_aS_a(0.2) > 0.75$, are specified in NBC Sentence 4.1.8.16.(8) to address the high levels of expected ground motions and the cyclic nature of those motions.

Clause (a): In regions of high seismicity, it is expected that the earthquake forces acting on the structure will generate relatively large moments in piles, drilled piers or caissons. These elements must therefore be designed and detailed to accommodate cyclic inelastic behaviour; Clause (a) requires such detailing when the element design moment is greater than 75% of the element’s moment capacity, calculated for the amount of axial load that is present.

Clause (b): Site Classes E and F, as defined in NBC Table 4.1.8.4.-A, comprise soft and very soft soils. When spread footings in regions of high seismicity are founded on such soils, they must be tied together to provide lateral restraint to prevent damage to the structural system immediately above the spread footings. Design force requirements for such ties are specified in NBC Sentence 4.1.8.16.(9).

NBC Sentence 4.1.8.16.(9)

225. Although there is no rational analysis available for the determination of the design forces for the ties specified in NBC Clauses 4.1.8.16.(6)(a) and 4.1.8.16.(8)(b), it is standard practice for such horizontal design forces to be proportional to the vertical load in the elements being connected by the ties. FEMA 302 specifies that the tie force be 0.25 times the short-period design spectral response acceleration times the maximum vertical load, while the more recent FEMA 368 reduces that multiplier from 0.25 to 0.10. NBC Sentence 4.1.8.16.(9) requires that the vertical load multiplier of the largest factored vertical load be 0.10 $I_E F_a S_a(0.2)$. Ties must be designed to carry the tie force in either compression or tension.

226. NBC Sentence 4.1.8.16.(9) also allows for the tie design force to be reduced or for ties to be omitted if it can be demonstrated that equivalent restraint can be provided by other means, e.g., as indicated in FEMA 368. Reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade are acceptable equivalent means of restraint; confinement by passive soil pressure against buried pile caps is not acceptable.

NBC Sentence 4.1.8.16.(10)

227. Although ground shaking, as described in terms of spectral response accelerations for a range of periods, is the normal earthquake-related hazard that affects the design of buildings and their foundations, earthquakes can cause other site hazards such as fault rupture, liquefaction, ground deformation and slope instability. Of these additional site hazards, liquefaction and its consequences, i.e., ground displacement and loss of soil strength and stiffness, have been major sources of building damage during past earthquakes. NBC Sentence 4.1.8.16.(10) requires that the potential for liquefaction and its consequences be evaluated and taken into account in the design of the structure and its foundations. A methodology for the evaluation of the potential for liquefaction is described by Youd et al.;[124] variations of that methodology have been proposed by Seed et al.[125] and Idriss and Boulanger.[126] All these methodologies use the PGA values given in NBC Table C-3 of Appendix C of Division B. The US National Science Foundation has struck a committee to review all data and recommend a consistent state of practice. The consequences of liquefaction to be considered should include an evaluation of post-earthquake total and differential re-consolidation settlements and lateral spread displacements. The Canadian Foundation Engineering Manual,[55] Ishihara and Yoshimine,[127] and Tokimatsu and Seed[128] describe methodologies for evaluating post-liquefaction settlement; procedures for estimating lateral spread displacements are described by Youd et al.,[129] Zhang et al.,[130] and Faris et al.,[131] among others. Due to the different approaches of the various procedures, it is recommended that several methodologies be carried out before arriving at a conclusion. It should also be noted that using the probabilistic ground motions in the NBC 2015 with these empirical and semi-empirical analysis procedures, which are based on deterministic ground motions, poses a difficulty that requires sound engineering judgement.

Site Stability (NBC Article 4.1.8.17.)

228. Procedures for seismic assessment of slope stability and displacements for non-liquefied ground conditions can be found in APEGBC.[132]

Elements of Structures, Non-structural Components and Equipment (NBC Article 4.1.8.18.)

229. Items that are attached to buildings, i.e., non-loadbearing structural elements, architectural components, mechanical equipment and electrical equipment, must be designed so that they neither fail nor become detached from the building during design earthquake ground motion and become a major threat to life safety. NBC Table 4.1.8.18. lists the categories of attached items, which include tanks and their contents when located within a building:

Structural components: Categories 1 to 6
Architectural components: Categories 7 to 10
Mechanical and electrical components, including tanks: Categories 11 to 17
Other components: Categories 18 to 21
Elevator equipment and rails: Category 22
Pallet racks: Categories 23 and 24.

230. The design requirements in NBC Article 4.1.8.18. are intended to ensure that attached components and their connections to the building retain their integrity during strong ground shaking. The design force equations and the values of the parameters in those equations are based on those contained in FEMA 368, which originated from a study done by Bachman et al.[133] Their adaptation for use in the NBC and the implications for design are described by McKevitt.[134] Specific requirements for elevators, pallet racks and glass have been added to the NBC 2015. Guidelines for the seismic risk reduction of attached components are given in CAN/CSA-S832, “Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings.”

NBC Sentence 4.1.8.18.(1)

231. Attached components need to be designed and detailed so that they retain their integrity and do not become detached from the structure when subjected to forces arising from the design level earthquake ground motion. In order to retain their integrity, they also must be able to accommodate the resulting component deflections as well as the earthquake-generated building deflections: e.g., interior wall panels must be able to accommodate the in-plane and out-of-plane interstorey drifts. Of equal importance is the design and detailing of the connections between the components and the building structure (NBC Sentence 4.1.8.18.(7) gives additional requirements regarding connections).

The component design force, V_p, which is to be distributed according to the distribution of mass, is given by:

$$V_p = 0.3 I_E F_a S_a(0.2) S_p W_p$$

where

$0.3I_EF_aS_a(0.2)$ is equivalent to the expected peak acceleration at the base of the building (this particular value is based on experience and is approximately equal to the corresponding factor used in FEMA 368),

S_p is the component response factor, described in detail below, which accounts for the nature of the element, its position in the building, and its dynamic properties in relation to those of the supporting structure,

W_p is the weight of the component, and

F_a is the short-period site coefficient used in the design of the building. When designing elements within existing buildings for which there is insufficient geotechnical information to determine F_a, then the maximum value of F(0.2) for the appropriate value of PGA_{ref} in NBC Table 4.1.8.4.-B may be used as an upper bound for any Site Class, including Site Class F.

The component response factor, S_p, is determined as follows:

$$S_p = C_p A_r A_x / R_p \text{ subject to } 0.7 \leq S_p \leq 4.0$$

The factors in the above expression are defined as follows:

C_p accounts for the risk associated with the failure of the component. Higher values are assigned to components that contain toxic or explosive materials in recognition of the consequences associated with the possible release of these materials. C_p has a value of 1.00 for ordinary components and of 1.50 for those containing toxic or explosive materials. The value of 0.70 assigned to Category 13 (flat bottom tanks attached directly to a floor at or below grade within a building) reflects the low risk of failure associated with such tanks.

A_r represents the dynamic amplification of the component relative to the position of its attachment to the building structure. It is a function of the ratio of the natural period of the component to the fundamental period of the building structure. Highest amplifications (2.50) occur when the two periods are similar; there is no amplification (A_r = 1.00) when they are far apart.

A_x represents the amplification of the acceleration from the base of the building structure to the height at which the component is attached. This factor is only dependent upon the height at which the component is attached, and is given by:

$$A_x = (1 + 2h_x/h_n)$$

R_p is the component response modification factor, which recognizes the energy dissipation capability of the component and its connection to the structure; it serves the same function as the reduction factor, R_dR_o. Values assigned to the different categories of components range from 1.00 to 5.00 and are based on experience from past earthquakes and on the judgment of engineers familiar with the components' behaviour.

The values of the factors C_p, A_r, and R_p for the 24 different categories of attached components are given in NBC Table 4.1.8.18.

NBC Sentence 4.1.8.18.(2)

232. Non-structural components attached to non-post-disaster buildings pose little risk to life safety in regions of low to moderate seismicity. Consequently, NBC Sentence 4.1.8.18.(2) exempts such components (i.e., Categories 7 to 22) when the importance-modified short-period design acceleration, $I_E F_a S_a(0.2)$, is less than 0.35. Category 6 components (horizontally cantilevered floors, balconies, beams, etc.) are also exempted in such situations because they are subject to vertical earthquake ground motions, which tend to be lower in amplitude than horizontal ones.

Precast concrete cladding panels are generally considered as non-loadbearing components that are subject to wind and earthquake loads in addition to carrying their own dead load. These types of panels and their connections must be designed and detailed so that they retain their integrity and do not become partly or completely detached from the structure during the design earthquake. They should be considered as Category 1 or 2 walls from NBC Table 4.1.8.18., as appropriate. Additional information on the seismic design of precast concrete cladding panels can be found in the CPCI Design Manual, "Precast and Prestressed Concrete."[135]

NBC Sentence 4.1.8.18.(3)

233. Categories 11 and 12 in NBC Table 4.1.8.18. (machinery, fixtures, equipment and tanks containing or not containing toxic or explosive materials) each have subcategories that differentiate components that are rigid and rigidly connected and those that are flexible or flexibly connected. The distinctions are significant in that each subcategory has different values of the dynamic amplification factor, A_r, and the response modification factor, R_p. Components that are rigid and rigidly connected have no dynamic amplification (A_r = 1.00) while those that are flexible or flexibly connected have substantial dynamic amplification (A_r = 2.50). On the other hand, rigid and rigidly connected components have minimal energy dissipation (R_p = 1.25) while those that are flexible or flexibly connected have significant energy dissipation (R_p = 2.50). Because of the significant differences in the values of these factors for the two subcategories, it is necessary to provide a clear way of distinguishing between the two. NBC Sentence 4.1.8.18.(3) establishes the fundamental period as being the distinguishing characteristic. If the fundamental period of a component and its connection is less than or equal to 1.5 s, then the component is classified as rigid and rigidly connected; if the period is greater than 0.06 s, then the component is classified as flexible or flexibly connected. The flexibility in the second subcategory may be due to flexibility in the component and/or in its connection to the structure. If it is not feasible to reliably determine the fundamental period, then it would be appropriate for the designer to compute the force V_p on the assumption that the component is flexible or flexibly connected, since that case results in the larger force.

NBC Sentence 4.1.8.18.(4)

234. In determining the component design force, V_p, for access floors (Category 9), it is necessary to include both the dead load of the access floor itself and the weight of permanent equipment attached to the access floor; the latter is to be not less than 25% of the floor live load. This minimum value of added weight is necessary to ensure that floors and connections are adequately designed in situations where the equipment that is initially installed is relatively light but where subsequent modifications could result in the installation of heavier equipment. Both the connection of the access floor to the structure and the anchorage of equipment mounted on the access floor need to be designed to take into account shear and overturning moment arising from the motion of the equipment. The possibility of overturning is particularly important when the equipment is relatively tall and slender because of the possible risk to life safety; in such instances the force V_p should be applied at 75% of the height of the equipment (rather than at the centre of mass) to ensure an adequate representation of overturning effects. It should also be noted that the full weight, W_p, should be included in the floor weight, W_i, in the determination of the base shear, V, in accordance with NBC Article 4.1.8.7.

NBC Sentence 4.1.8.18.(5)

235. When the mass of a single flexible element or of a tank and its contents is more than 10% of the mass of the supporting floor, then the tank interacts dynamically with the floor rather than simply being an appendage attached to the floor. In such situations, the lateral forces must be determined by an analysis that considers the tank or flexible element and the supporting structure as a dynamically coupled system.

NBC Sentence 4.1.8.18.(6)

236. For all categories of components except Category 6 (horizontally cantilevered floors, balconies, beams, etc.), the design force, V_p, is to be applied horizontally in the direction that is the most critical for design. In some cases, the critical direction may vary for different connections of the same component (e.g., electrical cable trays and piping). For Category 6, V_p is to be applied vertically, either up or down, whichever direction produces the most critical effect; component gravity loads are to be included.

NBC Sentence 4.1.8.18.(7)

237. As previously noted, the connections between the attached components and the supporting structure have an important role. They must be designed to transfer the attachment forces, V_p, and the gravity loads arising from support of the components. NBC Clauses 4.1.8.18.(7)(a) to (f) specify some important additional requirements that must be met.

Clause (a): Friction due to gravity loads cannot be used to provide resistance to seismic forces; the three-dimensional dynamic motion of a component during seismic response can include rocking and twisting about the vertical axis, which can cause the component to "walk." This type of movement has been observed in past earthquakes for equipment such as tanks and transformers. For the special case of large interconnected rooftop arrays of ballasted photovoltaic panels that have been retrofitted onto existing roofs, additional information can be found in SEAOC Report PV1,[136] which provides prescriptive requirements for the use of friction to provide seismic restraint under certain special conditions.

Clause (b): R_p for inherently non-ductile connections, such as the adhesive bonding of components to the surface of the structure, or power-actuated fasteners, such as nails or bolts, should be taken as 1.0 to reflect the lack of ductility.

Clause (c): Anchorage in concrete using cast-in-place anchors or post-installed anchors (such as expansion, undercut and epoxy anchors), where the depth of embedment is less than 8 times the nominal diameter of the anchors, is limited to an R_p of 1.5, which accounts for the limited ductility of such anchors. These types of anchors should be qualified by testing procedures similar to those outlined in ACI 355.2, "Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary," and CSA A23.3.

Clause (d): Shallow drop-in-type anchors described in ACI 355.2 and power-actuated fasteners, such as nails and studs in concrete, must not be used to resist cyclic tension loading imposed by seismic response, as these types of connections are unable to withstand this type of loading. Post-installed anchors are to be used for this application; they should be qualified for earthquake loading in accordance with ACI 355.2 or ACI 355.4, "Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary."

Clause (e): Where interior or exterior walls and appendages (i.e., Categories 1 to 3 in NBC Table 4.1.8.18.) are attached to the building structure at heights above the first floor, there is a significant risk to life safety associated with such components becoming dislodged or falling off the side of the building. To avoid this possibility, this Clause requires that the fasteners used to attach such components to a building be designed for forces larger than those used to design the components. When the body of the connection—which is the link between the fasteners at each end of the connection—is ductile, the design forces used for the body of the connection are to be the same as for the component. However, the fasteners at each end of the connection, such as bolts, welds and plates, must be designed for twice the nominal yield capacity of the body of the connection. When the body of the connection is not ductile or is non-existent, the factor C_p is increased to 2.0 and R_p is taken as 1.0, which results in an increased design force for the connection.

Clause (f): For a connection to be considered ductile, the body of the connection must be the yielding inelastic element and the attachment at the end of the body of the connection must be strong enough to remain elastic.

NBC Sentence 4.1.8.18.(8)

238. Although floors and roofs acting as diaphragms are listed as Category 4 in NBC Table 4.1.8.18., no values of the factors C_p, A_r and R_p are specified. These structural components are not to be designed using the provisions of NBC Article 4.1.8.18. but must meet the requirements of NBC Article 4.1.8.15.

NBC Sentence 4.1.8.18.(9)

239. The load V_p specified in NBC Sentence 4.1.8.18.(1) is to be used in analyzing components and their connections for the purpose of determining lateral deflections. When an elastic analysis is used for determining lateral deflections, the computed deflections must be multiplied by R_p in order to determine realistic values of the anticipated deflections. This is directly analogous to the requirement for the determination of building structure deflections in NBC Sentence 4.1.8.13.(2) because the component forces have been reduced by R_p to take into account the inelastic energy dissipation capacity of the component and its connection. If the connection and component have different values of R_p, such as is required by NBC Clause 4.1.8.18.(7)(e), then the higher of the two values shall be used as the multiplier.

NBC Sentence 4.1.8.18.(10)

240. The approach for designing components and their connections specified in NBC Article 4.1.8.18. assumes that components do not interact with the structure other than at connection points and that the design of the structure accounts for the forces imposed upon it by the components. It is therefore important that there be sufficient clearance or separation between attached components and the structure, based on the deflections calculated in NBC Sentence 4.1.8.18.(9), so that accidental interactions do not occur causing the transfer of unexpected forces to the structure itself. When the components are rigid walls or panels, NBC Clause 4.1.8.3.(6)(b) applies, which requires that such components be made part of the SFRS if there is insufficient separation to preclude interaction.

NBC Sentence 4.1.8.18.(11)

241. Suspended equipment (e.g., pipes, ducts and cable trays), if not isolated, can be damaged due to pounding against the structure or other pieces of equipment. Such damage can be prevented by using seismic restraints, such as sway bracing, to restrict the lateral motion of the suspended equipment. Such restraints must be designed to meet the force and displacement requirements specified in NBC Article 4.1.8.18. and they must be located so that they do not impose bending on the hanger rods used to suspend the equipment because such rods and their connections are only designed to carry tension forces. In particular, threaded rods are subject to brittle failure at the root of the thread when subjected to bending.

NBC Sentence 4.1.8.18.(12)

242. If suspended equipment is located so that it is isolated from other pieces of equipment and nearby walls (e.g., pendent lights), then it may be designed as a pendulum system, in which case the supporting chains or rods must be designed to support twice the weight of the suspended equipment and the deflection requirements of NBC Sentence 4.1.8.18.(9) must be met, unless there is sufficient clearance for the suspended equipment to swing 45° without impacting adjacent equipment or walls, as indicated in CAN/CSA-S832.

NBC Sentence 4.1.8.18.(13)

243. Pallet storage racks can be tall and heavily loaded, and they can pose a significant risk to life safety if not properly designed and maintained. It is important that they be designed for earthquake loading in accordance with the NBC. Interior structures, such as free-standing storage racks, that are at or below grade and surrounded by, but not otherwise connected to, the building structure should be analyzed either as separate structures or as Category 23 or 24 components from NBC Table 4.1.8.18. If analyzed as separate structures, they must be separated in accordance with NBC Sentence 4.1.8.14.(1)

and values of R_d and R_o must be appropriate for the chosen structural system. The seismic design provisions given in CSA A344, "User Guide for Steel Storage Racks," are based on the NBC 1995 but they do not specify seismic force modification values. An updated rack design standard is under development for possible inclusion in CSA S16; values of R_d and R_o are being proposed as well as alternative seismic analysis methods. Adequate resistance to lateral forces and inelastic deformation capacity must be provided throughout the height of the structure. Inelastic deformation of the storage racks typically occurs at the connections. Behaviour of the rack joints used in the design must be validated through physical testing. Additional information on the design and use of storage racks can be found in ANSI MH16.1, "Design, Testing and Utilization of Industrial Steel Storage Racks," FEMA 460, "Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public," and CSA A344. These documents need to be applied in a manner that is consistent with the requirements of the NBC 2015, while taking into account R_dR_o values, site properties, importance factors, etc. Other rack structures within a building, such as portable racks, cantilever racks, drive-in/drive-through racks and shelving, are not included in the scope of these documents.

244. Except as noted in Paragraph 243, earthquake effects for racks mounted on floors above grade must be determined using either the method specified in NBC Article 4.1.8.18. or the Linear Dynamic Analysis Method, which considers both the rack and the building structure. If the former method is used, the forces at the attachments to the structure must be accounted for in the design of the supporting structural elements.

245. The seismic design of free-standing racks requires specialized analysis and, while it relies on the same principles as the seismic design of a free-standing building, there are, of course, differences:

(a) Racks behave differently in the down-aisle and cross-aisle directions. In the down-aisle direction, the non-linear response is often expected to occur in the connections of the frame; the analysis must take this non-linear response into account based on physical testing of the specific rack connections to be used.

(b) Where the response of the rack will be modified by base plate non-linear behaviour, then the dynamic model must take this into account.

(c) The fundamental period of racks must not be evaluated using the formulas for the fundamental period of building structures in NBC Part 4 because they do not represent the dynamic behaviour of storage racks. Rack periods can be calculated using dynamic analysis, which accounts for the non-linear behaviour of the connections and will produce an upper limit time period specific to rack structures.

(d) As the product loads on a rack can vary greatly, the most unfavourable loading configuration must be considered in the seismic analysis (a fully loaded rack will not necessarily be the most critical loading condition). The seismic weight includes the dead load of the structure plus its normal operating contents, but not less than a minimum fraction of the design product load, depending on the direction and/or period. The seismic weight can be reduced by the dynamically active fraction of the load.

(e) Movement of the product load on a rack during seismic events has a force-limiting and/or -damping effect.

(f) Stability effects (P-delta effects and notional loads) must be considered.

(g) Drift limitations must be such that the total rotation imposed on the beam-to-column connections from earthquake effects plus gravity loads does not exceed a fraction of the rotation capacity of the connections determined from physical testing.

(h) The same R_d and R_o values as those for conventional construction should be used, except that higher values in the down-aisle direction can be considered, provided physical testing has shown that the moment connections can achieve stable energy dissipation.

(i) Displacement-based methods of analysis may be appropriate for the down-aisle direction. They can be used, provided they have been proven by testing and analysis.

(j) Racks higher than 6 m have not been tested; the principles noted above may not work for taller racks.

246. Information on the design of elevators and escalators can be found in ASME A17.1/CSA B44, "Safety Code for Elevators and Escalators." In applying this standard, the designer should use ground motion parameters that are consistent with those used in the NBC 2015.

247. The design of free-standing tanks is outside the scope of the NBC. Their design should be based on current industry-accepted practice and consensus design standards (API 620, "Design and Construction of Large, Welded, Low-Pressure Storage Tanks," API 650, "Welded Tanks for Oil Storage," ANSI/AWWA D100, "Welded Steel Tanks for Water Storage," ANSI/AWWA D110, "Wire- and Strand-Wound Circular, Prestressed Concrete Water Tanks," ANSI/AWWA D115, "Circular Prestressed Concrete Tanks with Circumferential Tendons," and FEMA 450-1, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1: Provisions"). In applying these industry standards to Canadian locations, the designer should use ground motion parameters consistent with those used in the NBC 2015.

NBC Sentences 4.1.8.18.(14) and (15)

248. Falling glass presents a grave hazard to life safety during an earthquake. Glass can fall out of its frame, or break and then fall out. NBC Sentence 4.1.8.18.(14) requires that the displacement of the frame holding the glass be calculated and the assembly be checked to see if the glass will fall out, or crack and fall out. Information on acceptance criteria for such testing can be found in AAMA 501.6, "Recommended Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System." NBC Sentence 4.1.8.18.(15) exempts the following glazing systems from the requirements of NBC Sentence 4.1.8.18.(14): glass assemblies with sufficient clearance between the frame and the glass as to permit the glass to move within the frame during an earthquake without breaking or falling out; glass panels installed in buildings in low risk areas; tempered glass installed in non-post-disaster buildings at low heights above potentially occupied areas; and glazing assemblies in which the glass is laminated and attached to the frame. Useful information on glass and earthquake deflections can be found in ASCE/SEI 7, FEMA 450-1, FEMA 450-2, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 2: Commentary," and FEMA P-750.

NBC Sentence 4.1.8.18.(16)

249. Elements of structures, non-structural components and equipment within a structure with a supplemental energy dissipation system must be designed in accordance with this Sentence. See Figure J-31.

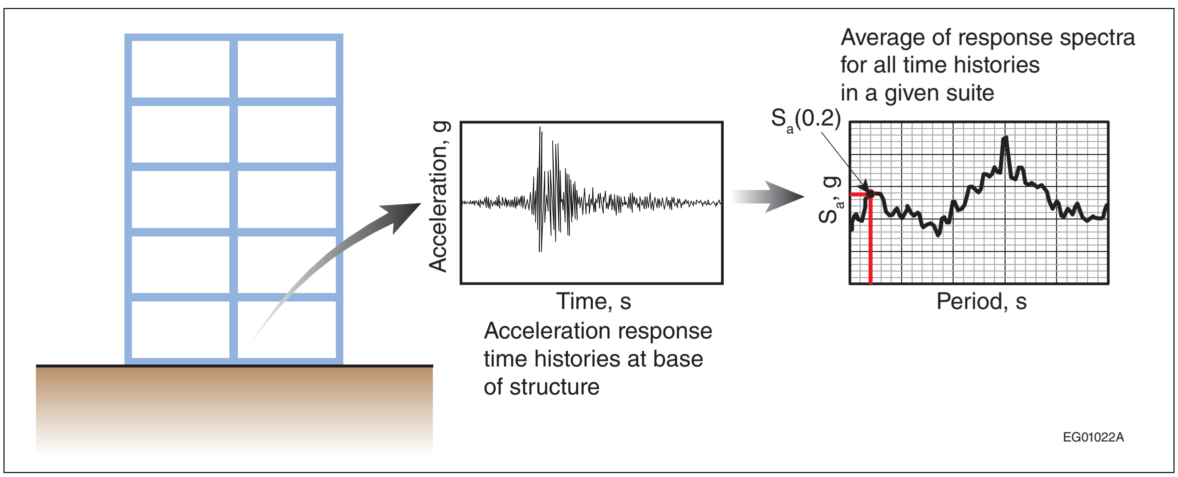


Figure J-31
Procedure for the design of elements of structures, non-structural components and equipment in a building with supplemental energy dissipation system

Seismic Design with Seismic Isolation (NBC Articles 4.1.8.19. and 4.1.8.20.)

250. Seismic isolation is a structural design concept that is widely used in many countries for the design of new buildings. This design concept is particularly suitable for buildings in regions of high seismicity, but is also suitable for certain types of buildings in regions of moderate or low seismicity, and can be particularly effective for buildings with irregularities. It can also be used for the seismic retrofit or upgrade of existing buildings. The seismic design of buildings using seismic isolation needs to be tailored to the particularities of each building, and it is strongly recommended that an independent peer review be carried out.

251. The fundamental goal of seismic isolation is to reduce the earthquake-induced forces and energy transmitted to the structure. This goal is achieved by interposing an isolation system with low horizontal stiffness between the substructure and the superstructure of a building. During an earthquake, lateral displacements occur primarily in the isolation system along the isolation interface; as such, the lateral loads transmitted to the structure and the relative lateral displacements of the structure are greatly reduced. The low horizontal stiffness of the isolation system results in a modified structure that has a fundamental lateral period that is much longer than if the same structure were on a fixed base. As a consequence, the first dynamic mode of the isolated structure involves the deformation of the isolation system only, while the superstructure above the isolation interface remains essentially undeformed. The higher modes of the structure, which would produce deformations in the superstructure, generally have very low modal participation factors, so their contribution to building deformations is minimal, even if the ground motion has high energy at the periods corresponding to those modes. In general, seismic isolation systems have isolator units that provide significant damping and thus do not require supplemental damping to function effectively. However, supplemental damping can be beneficial to suppress any possible resonance at the fundamental lateral period of the isolated structure.

252. Two approaches are commonly used to provide seismic isolation (Naeim and Kelly[137]). One approach uses elastomeric bearings, where the elastomer is made of rubber or rubber laminated with steel (or another material). Examples of elastomeric isolator units include lead rubber bearings with a lead core (see Figures J-32 and J-33), rubber bearings with supplemental energy-dissipating elements (dampers), and high-damping rubber bearings. In some cases, these isolator units are combined with sliding isolators (sliders), which typically include polytetrafluoroethylene (PTFE, commonly referred to as Teflon) discs that slide on stainless steel plates. A second approach uses a re-centering sliding system, such as a friction pendulum system (see Figure J-34), in which a special interfacial material slides on material such as stainless steel. In a friction pendulum system, an articulated slider within a bearing slides along a stainless steel concave surface during an earthquake, causing the superstructure to move in a pendulum motion. Other approaches for seismic isolation are available, and new ones are being developed by researchers and suppliers.

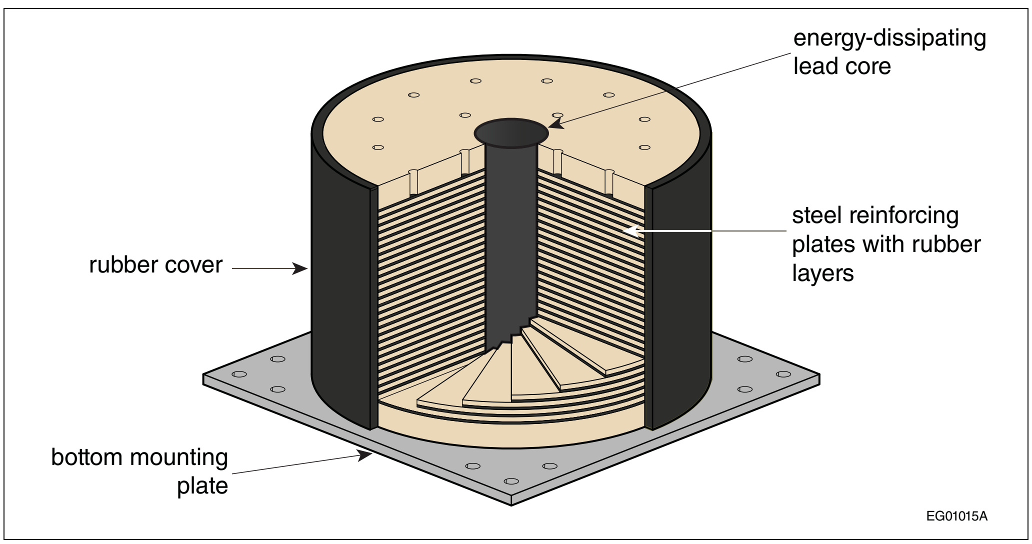


Figure J-32
Cut-away view of a lead rubber bearing

Figure J-33
Installation of a lead rubber bearing

Figure J-34
A triple-pendulum friction system (photo courtesy of Earthquake Protection Systems, California)

NBC Sentence 4.1.8.19.(1)

253. Figure J-35 illustrates many of the terms defined in this Sentence. Isolation systems can use one type of isolator unit or a combination of different types, e.g., non-re-centering sliding isolators together with elastomeric isolators. In some isolation systems, supplemental energy-dissipating devices or wind-restraint devices are included to supplement the isolator units.

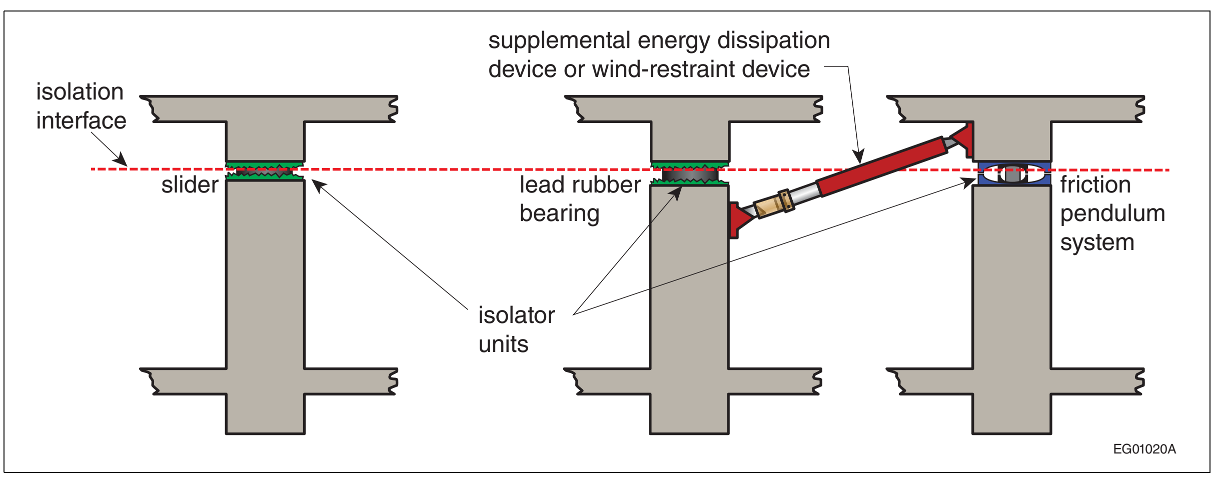


Figure J-35
Examples of isolator units in a seismically isolated structure

254. As illustrated in Figure J-36, the isolation interface (sometimes referred to as the isolation plane) can be located above grade within the superstructure, below grade within the basement (commonly just beneath the ground floor), or beneath the foundation of the basement. The isolation interface does not necessarily need to be on one horizontal plane; the levels of the isolator units may differ as long as the lateral Total Design Displacement (TDD) can be accommodated at the location of each isolator unit and the rigid diaphragm requirements discussed in Paragraph 263 are addressed. Special care needs to be exercised if the level of the isolation interface varies significantly.

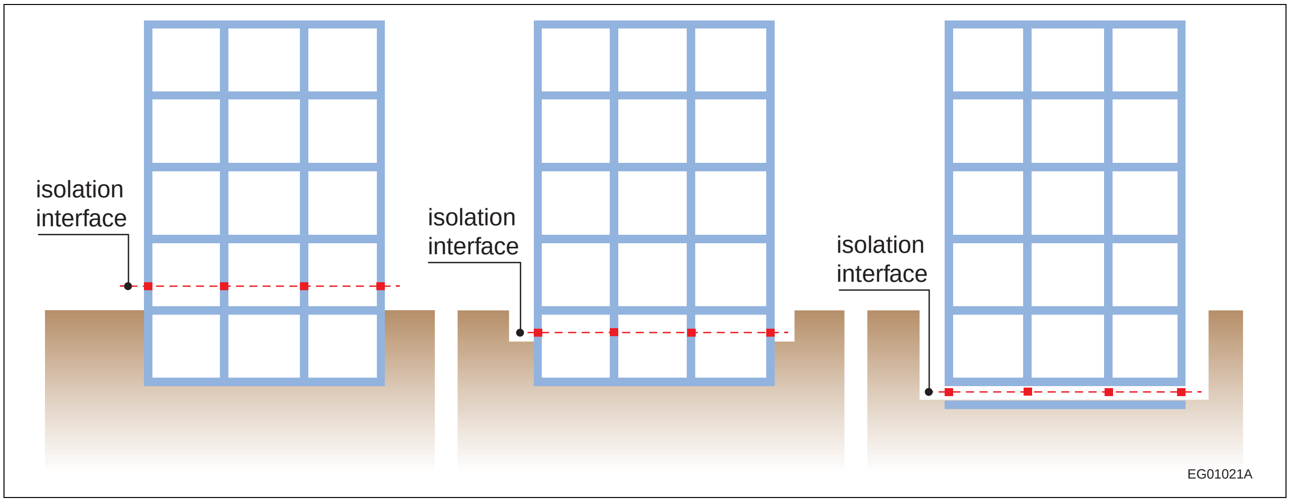


Figure J-36
Possible locations of the isolation interface

NBC Sentence 4.1.8.19.(2)

255. It is strongly recommended that a design review of the seismically isolated structure and its isolation system be carried out as outlined in NBC Note A-4.1.8.19.(2).

Clause (c): Detailed information on the seismic design of buildings with seismic isolation, including both theory and examples of practical applications, is given in FEMA P-751, "2009 NEHRP Recommended Seismic Provisions: Design Examples," and by Naiem and Kelly,[137] Kelly et al.,[138] and Kelly.[139] Useful information on the design of seismic isolated buildings can also be found in ASCE/SEI 7, ASCE/SEI 41 and FEMA P-750, "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures." Although the seismic isolation requirements in CSA S6, "Canadian Highway Bridge Design Code," and the AASHTO Guide Specifications for Seismic Isolation Design[140] are not specifically intended to apply to buildings, they are a useful complement to other information on seismic isolation.

NBC Sentence 4.1.8.19.(3)

256. A three-dimensional Non-linear Dynamic Analysis of a seismically isolated structure is required. It is strongly recommended that the soil–structure interface be modeled in the three-dimensional analysis, as variations in the foundation stiffness below the isolation interface can result in significantly different seismic demands on the structure above the isolation interface. It is important to carry out sensitivity analyses examining the soil–structure interface and the variation in the stiffness of the material supporting the foundation (e.g., soil or piles). The soil–structure interface must be modeled using geotechnical parameters obtained for the foundation design. Designers should consider carrying out sensitivity analyses for the lower- and upper-bound stiffness values of the soil. Typically, a first set of sensitivity analyses is performed that considers the lower-bound stiffness of the foundation combined with the lower-bound properties of the isolation system, and a second set of sensitivity analyses is performed that considers the upper-bound stiffness of the foundation combined with the upper-bound properties of the isolation system. However, in the interest of completeness, other combinations of foundation and isolation system properties should also be considered.

The three-dimensional analysis must take the following into consideration:

Clause (b): The equivalent viscous damping of the portions of the structure above and below the isolation interface, excluding the hysteretic damping provided by the isolation system at the isolation interface, must be less than or equal to 2.5% of the critical damping for the fundamental mode. The material properties of the structural system and the influence of the non-structural components should be considered in determining the value of the equivalent viscous damping. If the isolation interface is beneath the foundation, damping at the foundation–soil interface is generally not considered. A special study incorporating the independent peer review recommended in NBC Note A-4.1.8.19.(2) will have to be conducted if foundation damping needs to be considered.

Clause (c): Each isolator unit must be modeled with consideration of its non-linear force-deformation characteristics. It is strongly recommended that sensitivity analyses examining the variation in the non-linear force-deformation characteristics be carried out to account for the following: variation in the material properties of the isolator units as a result of fabrication tolerances; the effects of axial, shear or combined axial and shear loads on the force-deformation characteristics of the isolator units (e.g., the friction coefficient of sliding isolators may vary depending on the axial load); load-rate effects (velocity effects) on the properties of the isolator units (the properties can vary significantly between low-velocity and high-velocity loading); age effects resulting from changes in the properties of the isolator units over the design life of the structure (e.g., the properties of rubber change with age); effects of temperature variations on the properties of the isolator units (the properties at the maximum and minimum temperatures to which the isolators are expected to be exposed need to be considered); first-cycle effects (e.g., the force required for friction devices to achieve sliding in the first half cycle of loading may be higher than that required in subsequent cycles); and other effects that may be reported by the vendor or manufacturer of the isolator units. Designers should consider carrying out at least one set of analyses using lower-bound properties and one set of analyses using upper-bound properties, combining, as appropriate, some or all of the effects listed above.

Clause (d): Structural elements outside of the isolation system that are intended to behave elastically must be modeled with low-deformation properties. As required by NBC Sentence 4.1.8.3.(8), the effect of cracked sections in reinforced concrete and reinforced masonry elements must be accounted for. For concrete, refer to CSA A23.3 for properties consistent with service loads, and for reinforced masonry, refer to CSA S304 for properties consistent with seismic loads. If the variation in stiffness between uncracked and cracked sections is significant, designers should consider performing sensitivity analyses using upper- and lower-bound stiffness values. Non-structural elements that may affect the lateral or torsional stiffness of the structure should be included in the structural model.

NBC Sentence 4.1.8.19.(4)

257. **Clause (a):** For guidance on the selection and scaling of ground motion time histories for use in the three-dimensional Non-linear Dynamic Analysis, see the Commentary section on NBC Clause 4.1.8.12.(1)(b) (Paragraph 179) and the Appendix to this Commentary.

For structures with seismic isolation, the following requirements apply, which are different from those for conventional structures with an SFRS listed in NBC Table 4.1.8.9. or a similar one:

Subclause (b)(i): The use of the design response spectrum (i.e., the spectrum derived from the design spectral response acceleration values, S(T), defined in NBC Sentence 4.1.8.4.(9)) as the target spectrum for the selection and scaling of ground motion time histories is only permitted for Site Classes A, B and C.

Subclause (b)(ii): A site-specific design response spectrum must be used as the target spectrum for Site Classes D, E and F. The site-specific design response spectrum should be based on at least three soil investigations at different locations to account for variations in response across the plan area of the structure. In some cases, the values of spectral response acceleration in the site-specific design response spectrum may be much higher than the S(T) values specified in NBC Sentence 4.1.8.4.(9) over the period range of interest, and the use of seismic isolation may prove to be ineffective. The period of a seismically isolated structure may also fall within the resonance period range of the soil; in such cases, significant supplemental damping may be required for effective isolation.

Clause (c): A period range of $0.2T_1$ to $1.5T_1$ is used for scaling the ground motion time histories, where T_1 is the period of the isolated structure determined using the post-yield stiffness, k, of the isolation system in the horizontal direction under consideration. The post-yield stiffness of the isolation system is determined by the approach shown in Figure J-37, which can be used for isolation systems with different types of isolator units, such as lead rubber bearings and friction pendulum systems. The effective stiffness, k_{eff}, must not be used to determine T_1.

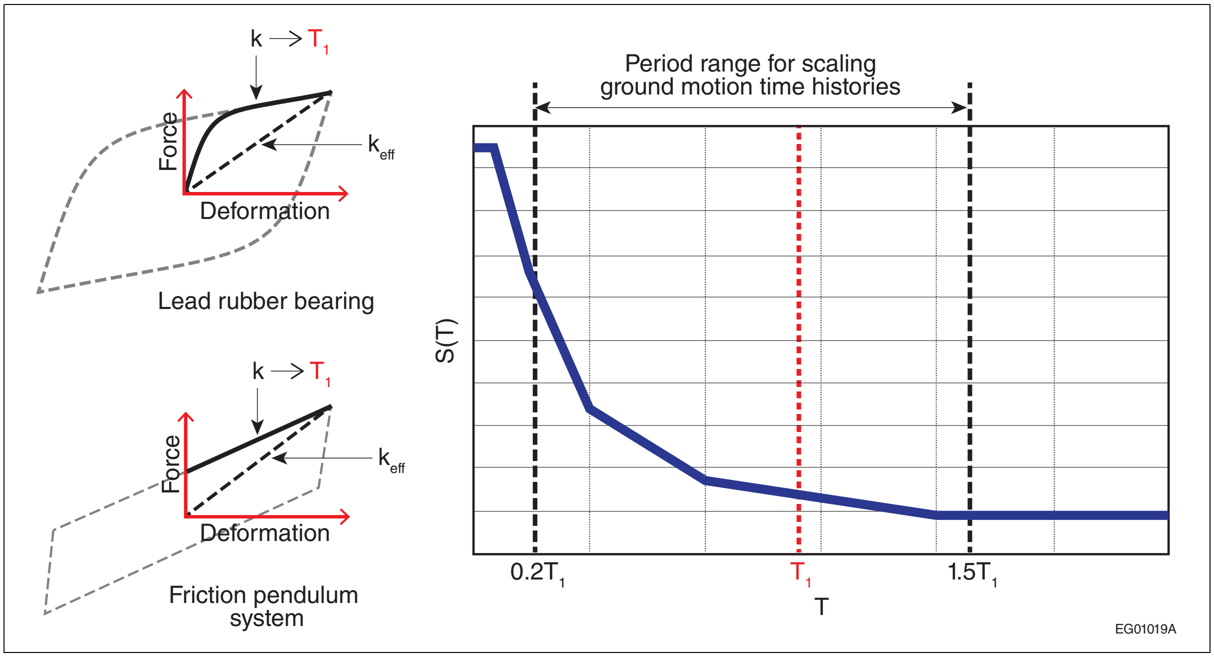


Figure J-37
Determining post-yield stiffness and the period range for scaling ground motion time histories

In general, a suite of ground motion records is used for the Non-linear Dynamic Analysis, where each record includes a pair of orthogonal horizontal ground motion components and the associated vertical ground motion component. However, if the seismic hazard in the period range of interest is affected by several different magnitude–distance scenarios or tectonic sources, it is recommended that one suite of ground motion records be used for each scenario or source. For example, for locations in southwestern British Columbia where three different sources may produce significant ground motions, three different suites should be used.

NBC Sentence 4.1.8.20.(1)

258. The structure above the isolation interface must be modeled as a fixed base structure to calculate its fundamental lateral period. It is recommended that a three-dimensional model of the fixed base structure be used in which pinned constraints are applied at the base and lower-bound stiffness values of the structural components are used (see Paragraph 256). The period of the seismically isolated structure, T_1, must be greater than three times the fundamental lateral period calculated for the fixed base structure to promote a first-mode-dominant response of the seismically isolated structure.

NBC Sentence 4.1.8.20.(2)

259. The isolation system must have a positive stiffness at deformations up to at least TDD. The centre of mass, CM, of the structure above the isolation interface can be determined from the three-dimensional model, and its coordinates in the horizontal x- and y-directions can be projected onto the structural diaphragm immediately above the isolation interface (see Figure J-38).

The TDD at the projected centre of mass can be determined from the analysis, and its components in the x- and y-directions, TDD_x and TDD_y, can then be calculated. The isolation system must produce a restoring force at TDD_x that is at least $0.025W_b$ greater than the restoring force at $0.5TDD_x$, where

W_b is the portion of W above the isolation interface, as shown in Figure J-38; the isolation system must also produce an analogous restoring force in the y-direction.

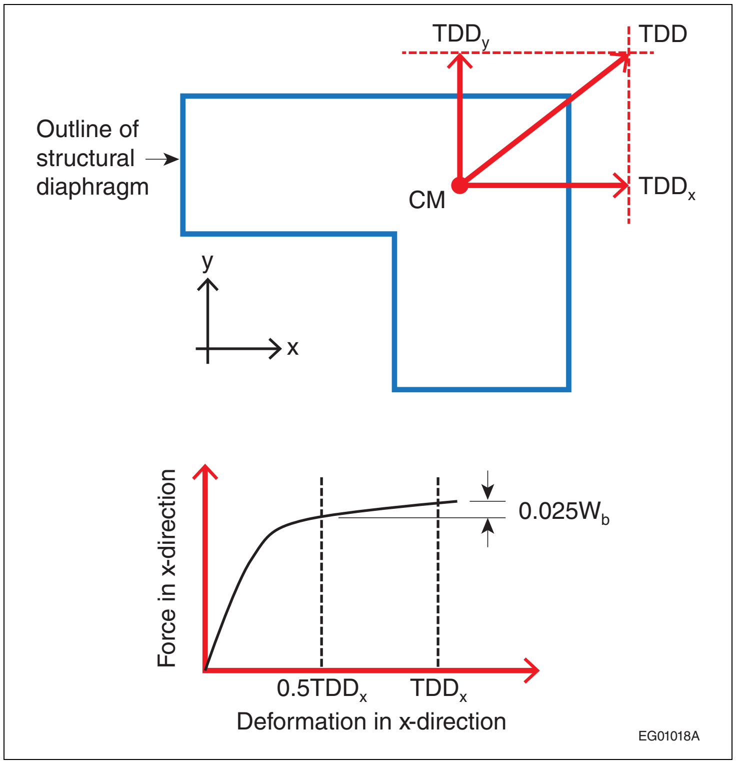


Figure J-38
Positive stiffness requirement for isolation systems

NBC Sentence 4.1.8.20.(3)

260. The values of forces and deflections (deformations) used in the design of structures with seismic isolation must be determined in accordance with the requirements of this Sentence, which are different from those for conventional structures with an SFRS listed in NBC Table 4.1.8.9. For conventional structures, the design values of forces and deflections are the mean values of the results of all Non-linear Dynamic Analyses (see the Commentary section on NBC Clause 4.1.8.12.(1)(b) in Paragraph 179 and the section titled Design Seismic Demand in the Appendix to this Commentary). In contrast, for seismically isolated structures, these design values are slightly higher than the mean values of the results because they are taken as the larger of the mean plus I_E times the standard deviation of the results and $\sqrt{I_E}$ times the mean, where I_E is the earthquake importance factor.

Furthermore, where more than one suite of ground motion records is needed to carry out analyses at sites affected by different magnitude–distance scenarios or tectonic sources, a set of design forces and deflections must be determined for each suite.

For a given suite of ground motion records, the maximum or minimum value of a specific parameter (e.g., force or bending moment) for a structural framing element or component of the isolation system is determined from analysis, and the critical combination of values so determined is used for design. For example, in designing a column, the combination of the maximum axial force, the maximum shear force and the maximum bending moment is considered for each horizontal direction. The design must satisfy the seismic demand (i.e., forces and deflections) for each suite of ground motion records; the maximum or minimum values of parameters determined for different suites do not need to be combined.

A column in the SFRS of a seismically isolated structure would be designed and detailed as follows:

1. For each suite of time history analyses, determine the maximum values of the axial force, the shear force and the bending moment as the larger of the mean plus I_E times the standard deviation and $\sqrt{I_E}$ times the mean.
2. Design for the results for each suite using $R_dR_o = 1.0$.
3. Detail for $R_d \geq 1.5$ where $I_E < 1.5$, and for $R_d \geq 2.0$ where $I_E = 1.5$.

NBC Sentence 4.1.8.20.(4)

261. The Non-linear Dynamic Analyses (including sensitivity analyses) of structures with seismic isolation are typically carried out using non-linear force-deformation characteristics of the isolator units that are based on information provided by the vendor. Prior consultation with the vendor is highly recommended to ensure that suitable isolator units are used.

The fundamental properties of the isolation system must be evaluated by prototype testing prior to its use. The purpose of these system characterization tests is to determine the properties of the isolator units and to verify the lower- and upper-bound properties used for the isolator units in the analyses.

At least two full-size prototypes of each predominant type and size of isolator unit must be tested to verify the force-deformation and damping characteristics used for the isolator units in the analysis. The isolator units used for prototype testing are not to be installed in the building. The prototype testing can be arranged by the building owner during the design process or, more typically, can be done by the contractor or vendor at the start of construction, usually immediately after fabrication drawings of the isolator units (based on contract specifications and drawings) are approved by the design engineer.

The prototype testing protocol is developed by the design engineer or by the manufacturer of the isolator units with the approval of the design engineer. The adequacy of the prototypes is evaluated in terms of positive incremental force-carrying capacity, variation in effective stiffness and damping, stability under vertical load, and deterioration. If a sacrificial wind-restraint system is used, its ultimate capacity must be established by testing. It is recommended that the design engineer or a designated representative witness the prototype testing of the isolator units and any other devices in the isolation system and review the testing report provided by the vendor or testing agency. Further information on requirements and acceptance criteria for prototype testing can be found in the references provided in Paragraph 255.

NBC Sentence 4.1.8.20.(5)

262. Once the prototype testing has been successfully completed and any adjustments to the design of the isolator units have been implemented, production testing of the isolator units is carried out. A representative sample of the isolator units to be installed in the building, including units of each predominant type and size, is tested prior to their installation. Codes and guidelines in some countries are moving towards requiring that every production unit be tested; such a level of testing should be considered, especially if it can be carried out in a cost-effective manner.

The force-deformation characteristics of the isolator units are determined from the results for each fully reversed cycle of loading in cyclic load tests and must meet acceptance criteria developed for each type of isolator unit. It is recommended that the design engineer or a designated representative witness the production testing of isolator units and review the testing report provided by the vendor or testing agency. The same production testing requirements apply to sacrificial wind-restraint and supplemental energy dissipation devices in the isolation system. Further guidance on the requirements and acceptance criteria for production testing can be found in the references provided in Paragraph 255.

NBC Sentence 4.1.8.20.(6)

263. It is important to provide a rigid diaphragm (such as a reinforced concrete slab) or an arrangement of horizontal structural elements (such as a grid of reinforced concrete beams or horizontal steel trusses) above the isolation system such that it is connected to the upper plates of the isolator units. The purpose of this rigid structural component is to ensure that the horizontal and torsional demands of the superstructure are appropriately distributed to all the isolator units.

NBC Sentence 4.1.8.20.(7)

264. Once the design forces have been determined according to the procedure of NBC Sentence 4.1.8.20.(3), all structural framing elements in the SFRS must be designed for elastic behavior under these forces by using $R_dR_o = 1.0$. However, to ensure that the structural framing elements have at least a modest level of ductility to allow them to withstand seismic demands higher than those determined from the analysis, they must be detailed as follows:

Clause (a): in accordance with the requirements for $R_d \geq 1.5$, for structures with $I_E < 1.5$, and

Clause (b): in accordance with the requirements for $R_d \geq 2.0$, for structures with $I_E = 1.5$.

These requirements apply to all the structural framing elements in the SFRS, both those below and above the isolation interface.

NBC Sentence 4.1.8.20.(8)

265. Since the SFRS in seismically isolated structures must remain elastic, there are no height restrictions for these structures. Note that tall structures with seismic isolation whose fixed-base period above the isolation interface is large may end up having a very large overall period, T_1, corresponding to a very low post-yield stiffness, k, of the isolation system (see Figure J-37). This low post-yield stiffness may result in excessively large lateral deformations at and above the isolation interface. Furthermore, tension in elastomeric isolator units (such as lead rubber bearings) and separation in sliding isolator units (such as friction pendulum systems) needs to be accounted for in the analysis of tall seismically isolated structures. The testing program for such structures must be expanded to appropriately evaluate these behaviours.

NBC Sentence 4.1.8.20.(9)

266. All isolator units must be designed to accommodate both the forces determined by the procedure in NBC Sentence 4.1.8.20.(3) and the TDD. It is critical to determine the TDD at the location of each of the isolator units, especially for those at the corners of the building plan where torsional effects can cause increased horizontal deformations.

NBC Sentence 4.1.8.20.(10)

267. The lateral displacement of the isolation system at the isolation interface due to the design wind load in any direction must not exceed the product of 1/500 times the least storey height of the structure above the isolation interface. For example, for a storey height of 3 000 mm, the lateral displacement due to the design wind load (including the gust effect factor and all other applicable factors) is limited to 6 mm. It is recommended that the yield force of elastomeric isolation systems and the force to cause sliding of re-centering sliding isolation systems be at least 1.5 times the factored design wind load, where wind is the principal load, at the base of the structure.

Structural Separation

268. The design of a seismically isolated structure at and above the isolation interface must accommodate the TDD in both orthogonal horizontal directions simultaneously. Where the isolation interface is below grade, a seismic gap or moat at and below grade around the structure is necessary to accommodate the TDD (see Figures J-39 to J-41).

Figure J-39
Seismic gap or moat to accommodate the TDD

Figure J-40
Cover plate over a seismic gap or moat

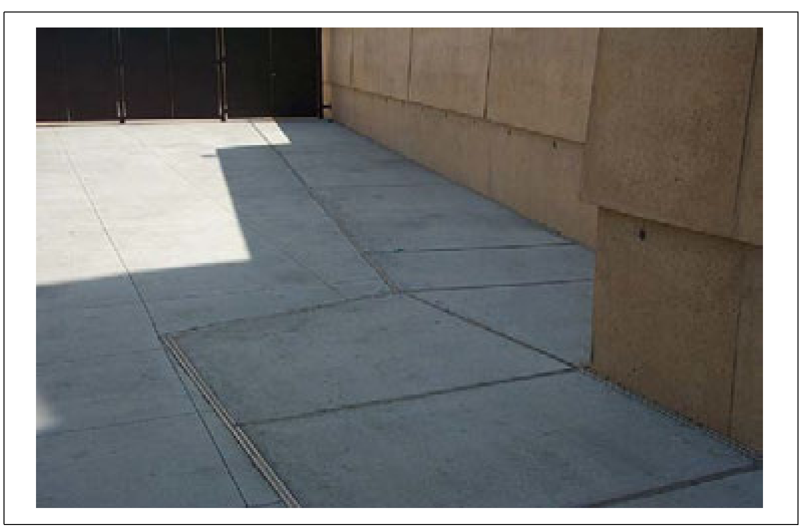


Figure J-41
Seismic gap or moat with a cover plate and finish installed

A seismically isolated structure must be separated from each adjacent structure as follows:

(a) at and above the isolation interface, by the absolute sum of their individual deflections, where the deflection of the seismically isolated structure is taken as the largest of the TDD values at all points on the seismically isolated structure facing the adjacent structure, and

(b) below the isolation interface, in accordance with NBC Sentence 4.1.8.14.(1).

Retaining walls and other fixed obstructions at and above the isolation interface must be separated from the seismically isolated structure and the isolation system by not less than TDD at the point of minimum separation.

Fire-Resistance Rating

269. All components of an isolation system that are located below a floor assembly required to have a fire-resistance rating must have a fire-resistance rating of no less than that required for the supported floor assembly. The fire-resistance rating of the components must also meet that required for loadbearing walls, columns and other gravity-bearing elements adjacent to the isolation system. The required fire-resistance rating can be achieved by installing sprinklers and/or commercially available fire blankets that are able to accommodate the movements of the isolator units.

Elements of Structures, Non-structural Components and Equipment

270. It is recommended that elements of structures, non-structural components and equipment within seismically isolated structures be designed in accordance with NBC Article 4.1.8.18., with a few exceptions, which are summarized in the following (see Figure J-42):

(a) For elements and components with a force amplification factor, A_r, of 1.00, the value of $S_a(0.2)$ in NBC Sentence 4.1.8.18.(1) is taken as the largest of the mean 5% damped floor spectral response acceleration values at periods of 0 to 0.2 s.

(b) For elements and components with $A_r = 2.50$, the value of $S_a(0.2)$ in NBC Sentence 4.1.8.18.(1) is taken as the largest of the mean 5% damped floor spectral response acceleration values at periods of 0.2 s to $1.5T_1$.

(c) The mean 5% damped floor spectral response acceleration values are determined by averaging the individual 5% damped floor response spectra at the centre of mass on the plane immediately above the isolation interface from all the Non-linear Dynamic Analyses.

(d) The value of F_a in NBC Sentence 4.1.8.18.(1) is taken as 1.00.

(e) All elements and components crossing the isolation interface must be designed to accommodate a displacement of 1.5TDD, where TDD is determined at the specific location of each element or component immediately above the isolation interface.

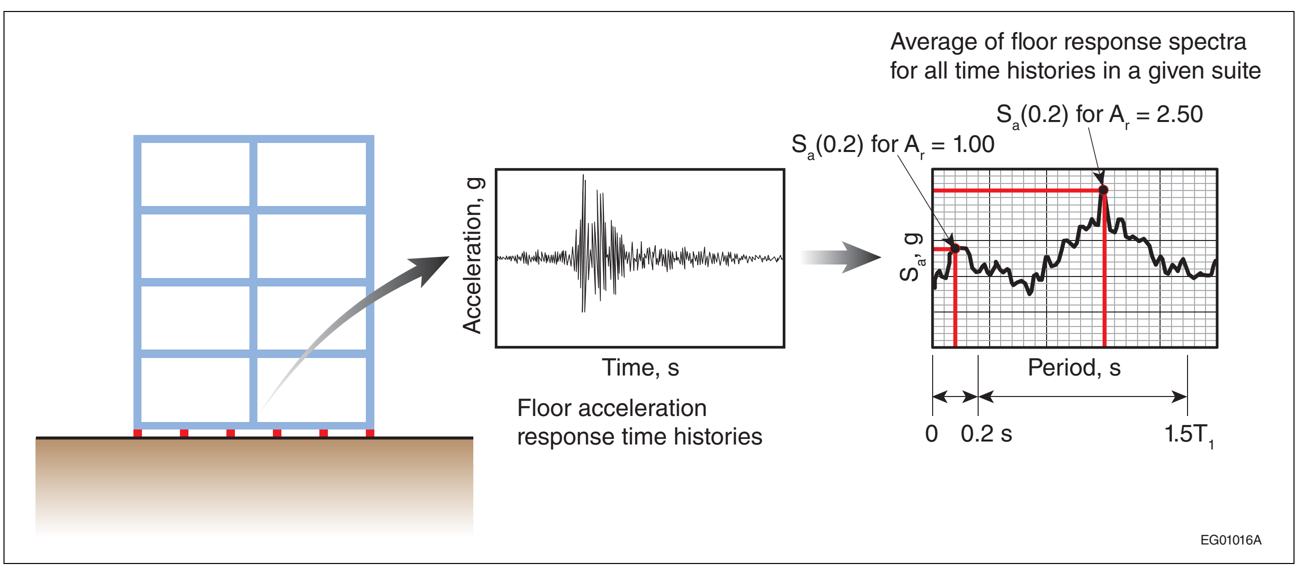


Figure J-42
Procedure for the design of elements of structures, non-structural components and equipment in a seismically isolated structure

The intent of these recommendations is to ensure that all building services that cross the isolation interface, such as elevators, stairs, and electrical, mechanical and HVAC services, remain functional after an earthquake.

271. The design of the seismically isolated structure and of the elements of structures, non-structural components and equipment within the structure must be evaluated for the demands of low-level earthquakes, i.e., for seismic forces that do not exceed the threshold force (the yield force for elastomeric isolation systems or the force to cause sliding for re-centering sliding systems). This evaluation should include the following:

- a conventional analysis of a fixed base structure, as discussed in Paragraph 258, using the threshold force as the base shear,
- confirmation that the seismic demands on the structural components are lower than those on the seismically isolated building, as discussed in Paragraph 260,
- confirmation that the displacements do not exceed the Code limits corresponding to the earthquake importance factor of the seismically isolated structure, and
- confirmation that the seismic demands on the elements of structures, non-structural components and equipment and their connections are lower than those outlined in Paragraph 270.

Seismic Design with Supplemental Energy Dissipation Devices (NBC Articles 4.1.8.21. and 4.1.8.22.)

272. Seismic design with supplemental energy dissipation devices is a structural design concept used widely in many countries for the design of new buildings. This design concept is particularly suited to buildings in regions of high seismicity, but is also suitable for certain types of buildings in regions of moderate or low seismicity. It can also be used for the seismic retrofit or upgrade of existing buildings. The seismic design of buildings using supplemental energy dissipation devices needs to be tailored to the particularities of each building, and it is strongly recommended that an independent peer review be carried out.

273. Supplemental energy dissipation devices, often referred to as dampers (even if damping is not the primary energy dissipation mechanism), may be inserted into a structural system with the express objective of reducing the seismic response of the overall building by absorbing or dissipating energy within the devices. The most common of these devices can be grouped into two categories, displacement-dependent and velocity-dependent, according to the primary energy dissipation mechanism. Displacement-dependent devices rely on relative displacements within the device for the dissipation of energy and are typically based on either metallic yielding or frictional sliding. Examples of such devices include metallic dampers, such as added damping and stiffness (ADAS) devices and triangular added damping and stiffness (TADAS) devices (see Figure J-43), lead extrusion dampers, and friction dampers (see Figures J-44 to J-46). Velocity-dependent devices dissipate energy in either solid or fluid-filled components within the devices and rely primarily on relative velocities within the devices for the dissipation of energy. Examples of such devices include viscous fluid dampers (see Figure J-47), viscoelastic dampers (see Figure J-48), and viscous walls (see Figure J-49). Other approaches are available, and new ones are being developed by researchers and suppliers.

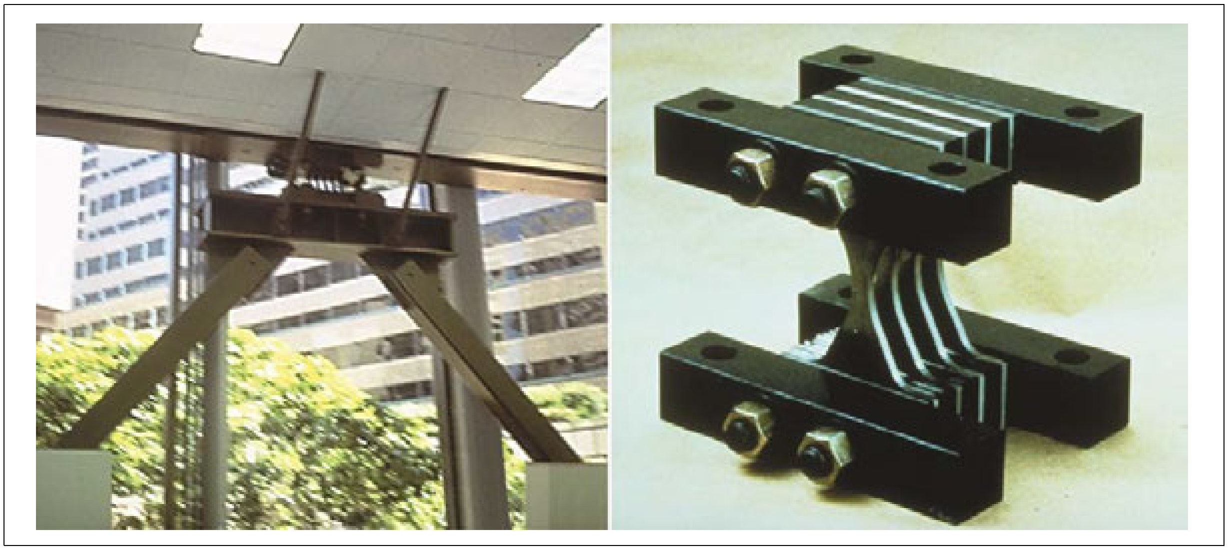


Figure J-43
Metallic dampers (TADAS and ADAS devices)

Figure J-44
Friction damper

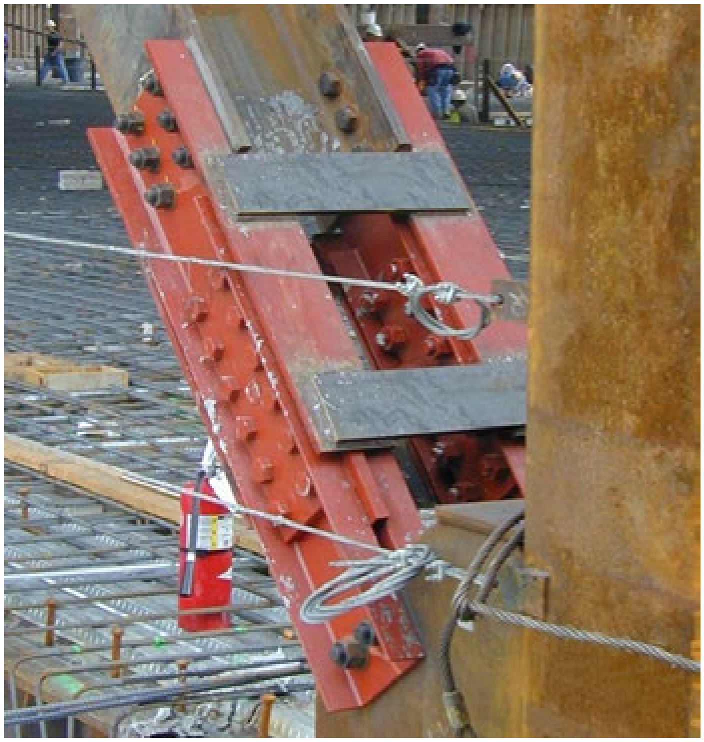


Figure J-45
Friction damper

Figure J-46
Friction damper

Figure J-47
Viscous fluid dampers

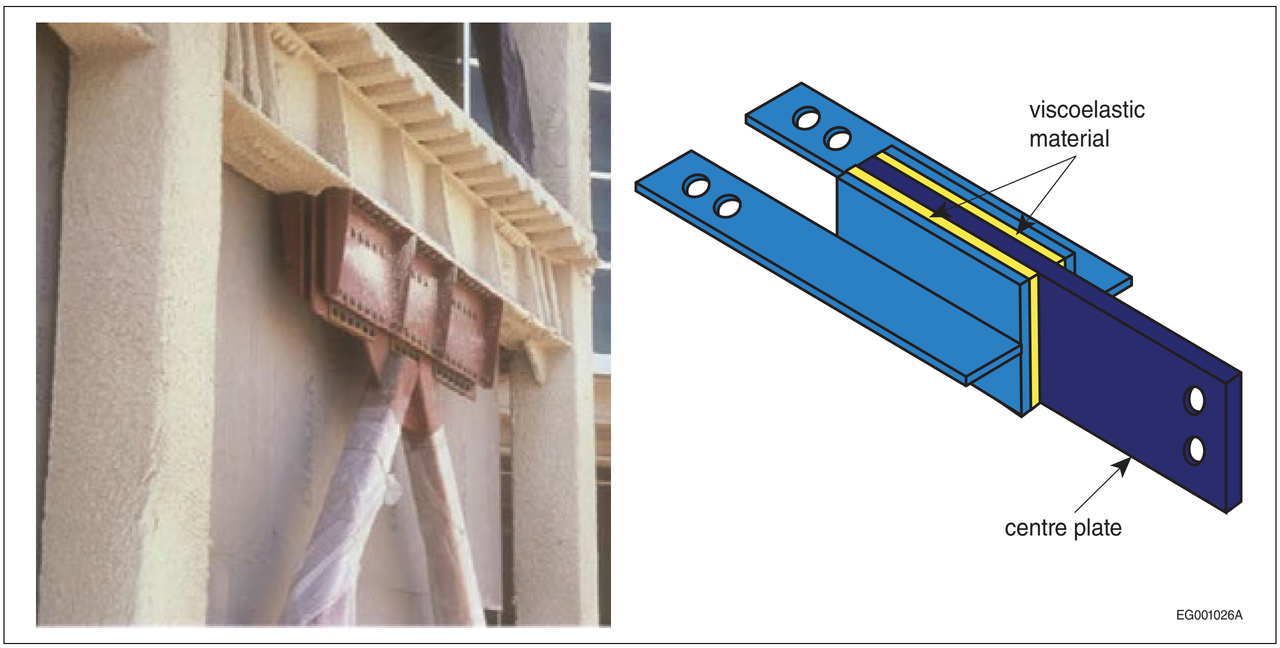


Figure J-48
Viscoelastic dampers

Figure J-49
Viscous wall

NBC Sentence 4.1.8.21.(1)

274. Figure J-50 depicts a supplemental energy dissipation system that includes viscous fluid dampers as the supplemental energy dissipation devices.

NBC Sentence 4.1.8.21.(2)

275. It is strongly recommended that a design review of the structure and the supplemental energy dissipation system be carried out as outlined in NBC Note A-4.1.8.21.(2).

Clause (c): Detailed information on the seismic design of buildings with supplemental energy dissipation devices, including both theory and examples of practical applications, is given by Hanson and Soong,[141] Kelly,[142] and Anderson et al.[143] Useful information on the design of buildings with supplemental energy dissipation devices can also be found in ASCE/SEI 7, ASCE/SEI 41, FEMA P-750, and FEMA P-751.

NBC Sentence 4.1.8.21.(3)

276. Supplemental energy dissipation devices are commonly used to provide additional damping in seismically isolated buildings, as illustrated in Figure J-35 in the Commentary section on NBC Sentence 4.1.8.19.(1) (starting at Paragraph 253). In such cases, the design displacements, velocities, accelerations and the resulting demands on the devices must be determined in accordance with NBC Sentence 4.1.8.20.(3).

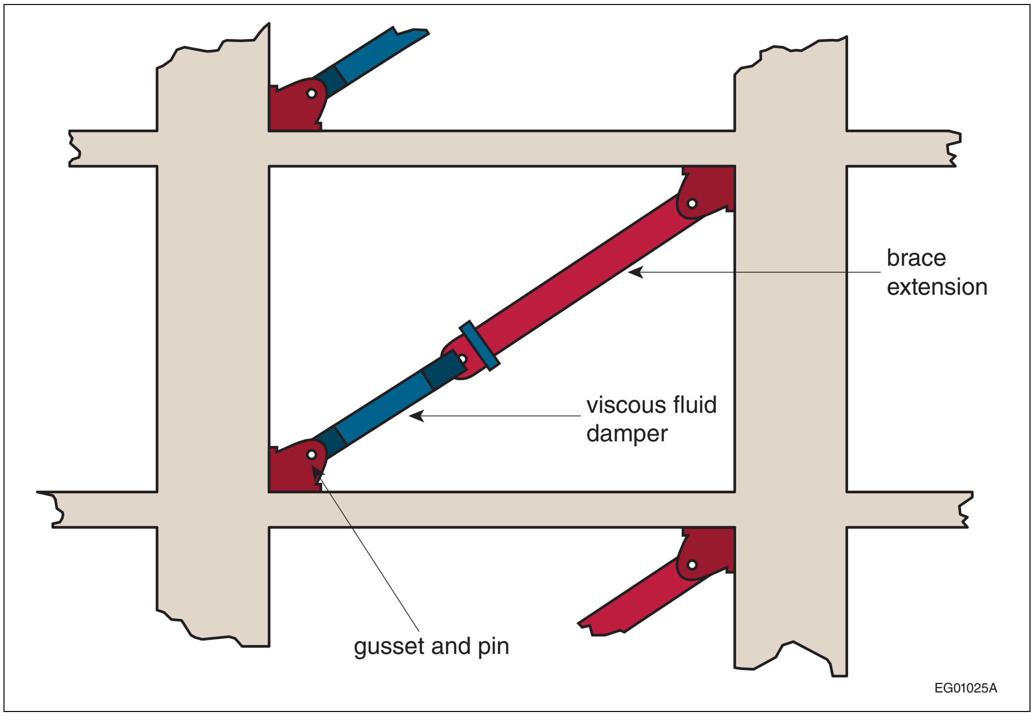


Figure J-50
Supplemental energy dissipation system including viscous fluid dampers

NBC Sentence 4.1.8.21.(4)

277. A three-dimensional Non-linear Dynamic Analysis of a structure with a supplemental energy dissipation system is required. Non-linear force-deformation characteristics must be modeled for each structural element of the SFRS that is intended to behave inelastically (i.e., with $R_d > 1.0$). Typically, the non-linear force-deformation characteristics of these elements are well defined, and sensitivity analysis to account for variation in the properties of the elements is not required.

It is strongly recommended that the soil–structure interface be modeled in the three-dimensional analysis, as variations in the foundation stiffness can result in significantly different seismic demands on the structure and the supplemental energy dissipation devices. It is important to carry out sensitivity analyses examining the soil–structure interface and the variation in the stiffness of the material supporting the foundation (e.g., soil or piles). The soil–structure interface must be modeled using geotechnical parameters obtained for the foundation design. Designers should consider carrying out sensitivity analyses for the lower- and upper-bound stiffness values of the soil. Typically, a first set of sensitivity analyses is performed that considers the lower-bound stiffness of the foundation combined with the lower-bound properties of the supplemental energy dissipation system, and a second set of sensitivity analyses is performed that considers the upper-bound stiffness of the foundation combined with the upper-bound properties of the supplemental energy dissipation system. However, in the interest of completeness, other combinations of foundation and supplemental energy dissipation system properties should be considered.

The three-dimensional analysis must take the following into consideration:

Clause (c): The equivalent viscous damping of the structure, excluding the damping provided by the supplemental energy dissipation devices, must be less than or equal to 2.5% of the critical damping for the fundamental mode. The material properties of the structural system and the influence of the non-structural components should be considered in determining the value of the equivalent viscous damping.

Clause (d): Each supplemental energy dissipation device must be modeled with consideration of its non-linear force-deformation (for displacement-dependent devices) or force-velocity (for velocity-dependent devices) characteristics. It is strongly recommended that sensitivity analyses examining the variation in the non-linear force-deformation or force-velocity characteristics be carried out to account for the following: variation in the material properties of the devices as a result of fabrication tolerances; load-rate effects (velocity effects) on the properties of the devices (the properties can vary significantly between low-velocity and high-velocity loading); age effects resulting from changes in the properties of the devices over the design life of the structure; effects of temperature variations on the properties of the devices (the properties at the maximum and minimum temperatures to which the devices are expected to be exposed need to be considered); first-cycle effects (for example, the force required for friction devices to achieve sliding in the first half cycle of loading may be higher than that required in subsequent cycles); and other effects that may be reported by the vendor or manufacturer of the devices. Designers should consider carrying out at least one set of analyses using lower-bound properties and one set of analyses using upper-bound properties, combining, as appropriate, some or all of the effects listed above and the considerations listed in NBC Sentence 4.1.8.22.(7).

Clause (e): Structural elements outside of the supplemental energy dissipation system that are intended to behave elastically must be modeled with low-deformation properties. As required by NBC Sentence 4.1.8.3.(8), the effect of cracked sections in reinforced concrete and reinforced masonry elements must be accounted for. For concrete, refer to CSA A23.3 for properties consistent with service loads, and for reinforced masonry, refer to CSA S304 for properties consistent with seismic loads. If the variation in stiffness between uncracked and cracked sections is significant, designers should consider performing sensitivity analyses using upper- and lower-bound stiffness values. Non-structural elements that may affect the lateral or torsional stiffness of the structure should be included in the structural model.

NBC Sentence 4.1.8.21.(5)

278. **Clause (a):** For guidance on the selection and scaling of ground motion time histories for use in the three-dimensional Non-linear Dynamic Analysis, see the Commentary section on NBC Clause 4.1.8.12.(1)(b) (Paragraph 179) and the Appendix to this Commentary.

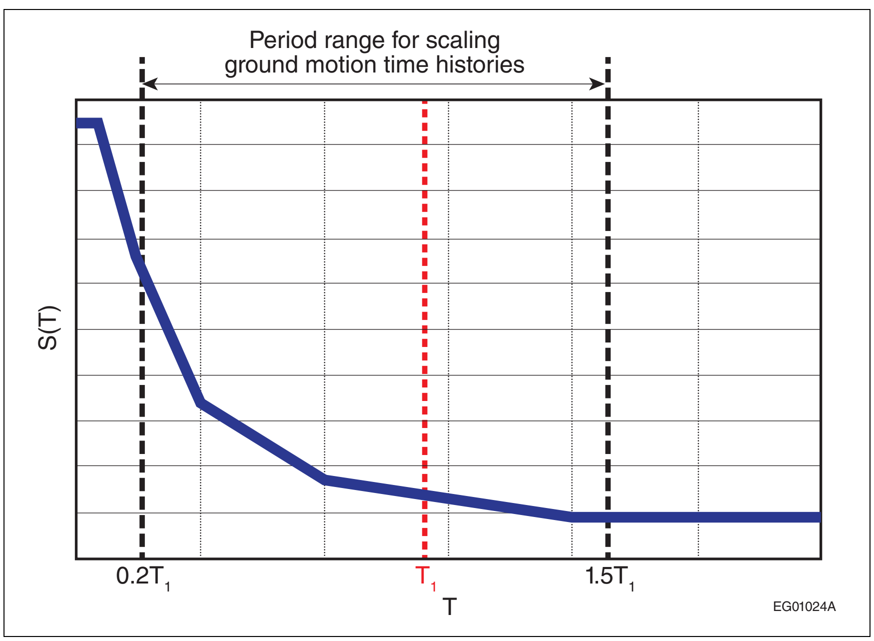


Figure J-51
Period range for scaling ground motion time histories

For structures with a supplemental energy dissipation system, the following requirement applies, which is different from the requirement for conventional structures with an SFRS listed in NBC Table 4.1.8.9. or a similar one:

Clause (c): Figure J-51 illustrates the period range of $0.2T_1$ to $1.5T_1$ that is used for scaling the ground motion time histories, where T_1 is the fundamental lateral period of the structure with the supplemental energy dissipation system. The value of T_1 is determined using a three-dimensional model of the structure that incorporates the added stiffness of the supplemental energy dissipation system.

In general, a suite of ground motion records is used for the Non-linear Dynamic Analysis, where each record includes a pair of orthogonal horizontal ground motion components and the associated vertical ground motion component. However, if the seismic hazard in the period range of interest is affected by several different magnitude–distance scenarios or tectonic sources, it is recommended that one suite of ground motion records be used for each scenario or source. For example, for locations in southwestern British Columbia where three different sources may produce significant ground motions, three different suites should be used.

NBC Sentence 4.1.8.22.(1)

279. The values of forces and deflections (deformations) used in the design of structures with a supplemental energy dissipation system must be determined in accordance with the requirements of this Sentence, which are different from those for conventional structures with an SFRS listed in NBC Table 4.1.8.9. For conventional structures, the design values of forces and deflections are the mean values of the results of all Non-linear Dynamic Analyses (see the Commentary section on NBC Clause 4.1.8.12.(1)(b) in Paragraph 179 and the section titled Design Seismic Demand in the Appendix to this Commentary). In contrast, for structures with a supplemental energy dissipation system, these design values are slightly higher than the mean values of the analysis results because they are taken as the larger of the mean plus I_E times the standard deviation of the analysis results and $\sqrt{I_E}$ times the mean, where I_E is the earthquake importance factor.

Furthermore, where more than one suite of ground motion records is needed to carry out analyses at sites affected by different magnitude–distance scenarios or tectonic sources, a set of design forces and deflections must be determined for each suite.

For a given suite of ground motion records, the maximum or minimum value of a parameter (e.g., force or bending moment) for a structural framing element or supplemental energy dissipation device is determined from analysis, and the critical combination of values so determined is used for design. For example, in designing a column, the combination of the maximum axial force, the maximum shear force and the maximum bending moment is considered for each horizontal direction. The design must satisfy the seismic demand (i.e., forces and deflections) for each suite of ground motion records; the maximum or minimum values of parameters determined for different suites do not need to be combined.

A column in the SFRS of a structure with a supplemental energy dissipation system would be designed and detailed as follows:

1. For each suite of time history analyses, determine the maximum values of the axial force, the shear force and the bending moment as the larger of the mean plus I_E times the standard deviation and $\sqrt{I_E}$ times the mean.
2. Where all the SFRS elements remain elastic (with $R_d = 1.0$),
   (a) design for the results for each suite using $R_dR_o = 1.0$, and
   (b) detail for $R_d \geq 1.5$.
3. Where some of the SFRS elements are allowed to yield (with $R_d > 1.0$),
   (a) design for the results for each suite by modeling the yielding SFRS elements in accordance with NBC Clause 4.1.8.21.(4)(b) and the other SFRS elements using $R_dR_o = 1.0$, and
   (b) detail for the R_d of the yielding SFRS elements.

NBC Sentence 4.1.8.22.(2)

280. The interstorey deflections in structures with a supplemental energy dissipation system must conform to the limits stated in NBC Sentence 4.1.8.13.(3) for conventional structures with an SFRS listed in NBC Table 4.1.8.9.

NBC Sentence 4.1.8.22.(3)

281. The Non-linear Dynamic Analyses (including sensitivity analyses) of structures with a supplemental energy dissipation system are typically carried out using non-linear force-deformation and force-velocity characteristics of the supplemental energy dissipation devices that are based on information provided by the vendor. Prior consultation with the vendor is highly recommended to ensure that suitable supplemental energy dissipation devices are used.

The fundamental properties of the supplemental energy dissipation system must be evaluated by prototype testing prior to its use. The purpose of these system characterization tests is to determine the properties of the supplemental energy dissipation devices and to verify the lower- and upper-bound properties used for the devices in the analyses.

At least two full-size prototypes of each predominant type and size of supplemental energy dissipation device must be tested to verify the force-deformation, force-velocity and damping characteristics used for the devices in the analysis. The devices used for prototype testing are not to be installed in the building. The prototype testing can be arranged by the building owner during the design process or, more typically, can be done by the contractor or vendor at the start of construction, usually immediately after fabrication drawings of the devices (based on contract specifications and drawings) are approved by the design engineer.

The prototype testing protocol is developed by the design engineer or by the manufacturer of the devices with the approval of the design engineer. The adequacy of the prototypes is evaluated in terms of force-carrying capacity, variation in damping, stability under vertical load, and deterioration. It is recommended that the design engineer or a designated representative witness the prototype testing of the supplemental energy dissipation devices and review the testing report provided by the vendor or testing agency. Further information on requirements and acceptance criteria for prototype testing can be found in the references provided in Paragraph 275 and in the AASHTO Guide Specifications for Seismic Isolation Design.[140]

NBC Sentence 4.1.8.22.(4)

282. Once the prototype testing has been successfully completed and any adjustments to the design of the supplemental energy dissipation devices have been implemented, production testing of the devices is carried out. A representative sample of the devices to be installed in the building, including devices of each predominant type and size, is tested prior to their installation. It is recommended that every device be tested, unless it can be shown by other means that their properties meet the requirements of the project specifications.

The force-deformation and force-velocity characteristics of the supplemental energy dissipation devices are determined from the results for each fully reversed cycle of loading in cyclic load tests and must meet acceptance criteria developed for each type of supplemental energy dissipation device. It is recommended that the design engineer or a designated representative witness the production testing of the devices and review the testing report provided by the vendor or testing agency. Further guidance on the requirements and acceptance criteria for production testing can be found in the references provided in Paragraph 275 and in the AASHTO Guide Specifications for Seismic Isolation Design.[140]

NBC Sentence 4.1.8.22.(5)

283. The pins, bolts, gusset plates, brace extensions and other components that connect the supplemental energy dissipation devices to the structure must remain elastic for the design forces determined from the Non-linear Dynamic Analyses in accordance with NBC Sentence 4.1.8.22.(1).

NBC Sentence 4.1.8.22.(6)

284. This Sentence provides two approaches for the design of structures with a supplemental energy dissipation system. Once the design forces have been determined according to the procedures in NBC Sentences 4.1.8.21.(4) and 4.1.8.22.(1), the structural framing elements in the SFRS are designed using one of the following approaches:

Clause (a): In the preferred and recommended approach specified in this Clause, all the structural framing elements are designed to remain elastic under the design forces by using $R_dR_o = 1.0$. However, to ensure that the structural framing elements have at least a modest level of ductility to allow them to withstand higher seismic demands than those determined from the analysis, all the elements are detailed in accordance with the requirements for $R_d \geq 1.5$.

Clause (b): In the alternative approach specified in this Clause, the design allows some of the structural framing elements in the SFRS—those with $R_d > 1.0$—to exceed the elastic range under the design forces. The structural framing elements that are intended to yield are modeled in accordance with NBC Clause 4.1.8.21.(4)(b), and all other structural framing elements are modeled as elastic by using $R_dR_o = 1.0$. To ensure that the structural framing elements will have adequate ductility in the event the actual distribution of forces results in inelastic demands on some of the elements intended to remain elastic, all the elements are detailed in accordance with the requirements for the R_d of the yielding elements.

NBC Sentence 4.1.8.22.(7)

285. This Sentence lists some of the considerations to be taken into account in the design of the supplemental energy dissipation system. Further information on such considerations can be found in the references provided in Paragraph 275.

NBC Sentence 4.1.8.22.(8)

286. This Sentence requires that means of access be provided for post-earthquake inspection and removal for replacement of all supplemental energy dissipation devices. The means of access enables the replacement of damaged devices, including those whose force-displacement or force-velocity characteristics are found to be different from the design characteristics and those that were intended to be damaged and replaced. The intent of replacing damaged devices as necessary is to provide a post-earthquake structure that meets the design intent of the original pre-earthquake structure.

Appendix

Selection and Scaling of Ground Motion Time Histories

1. General

This Appendix to Paragraphs 179 and 181 presents guidelines for the selection and scaling of ground motion time histories to be used in the dynamic time history analysis of structures, as prescribed by the National Building Code of Canada (NBC). Figure 1 outlines the steps involved and their corresponding Appendix sections.

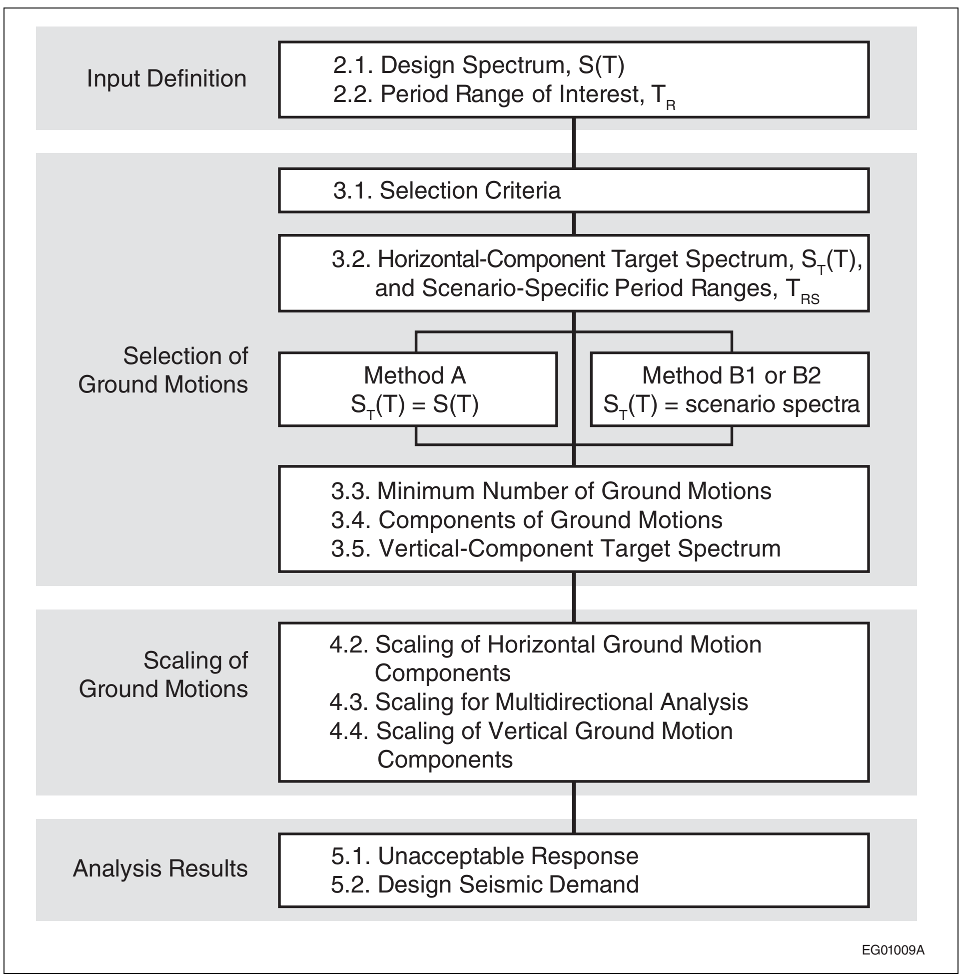


Figure 1
Steps in the selection and scaling of ground motions for seismic analysis

2. Design Spectrum and Period Range

2.1. Design Spectrum, S(T)

The design spectrum, S(T), which provides the overall target for ground motion selection and scaling, is as specified in the NBC 2015, for periods, T, greater than or equal to 0.5 s; for periods less than 0.5 s, S(T) can be obtained as follows, using linear interpolation for intermediate values of T:

$S(T) = F(PGA)PGA$ for $T = 0$ s,
$S(T) = F(0.05)S_a(0.05)$ for $T = 0.05$ s,
$S(T) = F(0.1)S_a(0.1)$ for $T = 0.1$ s,
$S(T) = F(0.2)S_a(0.2)$ for $T = 0.2$ s, and
$S(T) = F(0.3)S_a(0.3)$ for $T = 0.3$ s,

where

PGA is the peak ground acceleration;
F(PGA), F(0.05), F(0.1), F(0.2), and F(0.3) are the site coefficients at T = 0, 0.05, 0.1, 0.2, and 0.3 s, respectively; and
$S_a(0.05)$, $S_a(0.1)$, $S_a(0.2)$ and $S_a(0.3)$ are the 5% damped uniform hazard spectral response accelerations for periods T = 0.05, 0.1, 0.2 and 0.3 s, respectively.

Values of F(0.05), F(0.1) and F(0.3) are given in Paragraph 181; values of $S_a(0.05)$, $S_a(0.1)$ and $S_a(0.3)$ can be found on the Earthquakes Canada Web site using the “Hazard Calculator.”

Examples of design spectra with modifications in the short-period range are given in Figure 2.

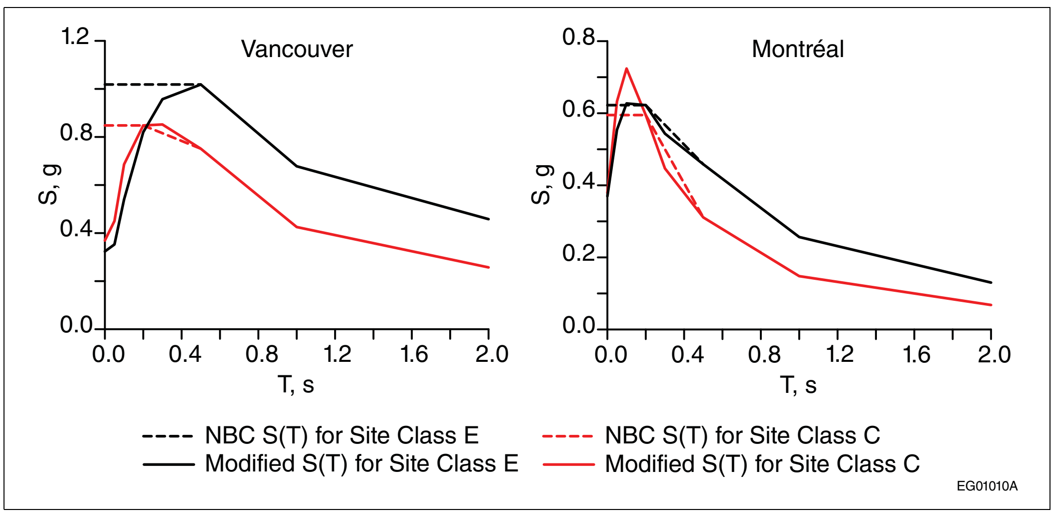


Figure 2
Examples of design spectra with modifications in the short-period range for Site Classes C and E in Vancouver and Montréal

For seismically isolated structures located on Site Class D, E, or F, a site-specific response spectrum should be used as the design spectrum, as noted in NBC Clause 4.1.8.19.(4)(b).

2.2. Period Range, T_R

For the purposes of ground motion selection and scaling, a period range, T_R, should be defined that covers the periods of the vibration modes that significantly contribute to the building’s dynamic response, either in the translational direction and/or in torsion. The upper-bound period, T_{max}, must be greater than or equal to twice the first-mode period, but not less than 1.5 s; the lower-bound period, T_{min}, should be established such that the range of periods from lowest to highest includes at least the periods of the modes that are necessary to achieve 90% mass participation, but not more than 0.15 times the first-mode period (see Figure 3). The dynamic properties of the building should be obtained from the structural model used for the time history analysis.

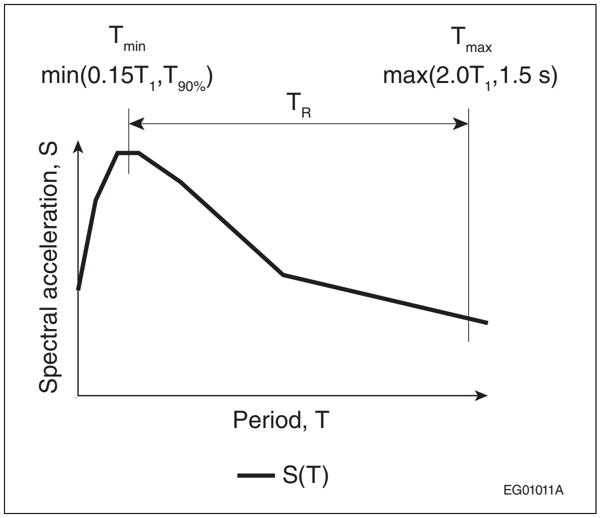


Figure 3
Period range, T_R

When the analysis is performed on a two-dimensional structural model, the upper- and lower-bound periods should be determined using the periods obtained in the direction considered. When the analysis is performed on a three-dimensional structural model, either with ground motion components in only one horizontal direction or with pairs of orthogonal horizontal ground motion components, the upper-bound period should be based on the longest first-mode period in the two orthogonal directions and the lower-bound period should be established to include the periods of the modes necessary to achieve 90% mass participation in each orthogonal direction, without exceeding 0.15 times the shortest first-mode period in the two orthogonal directions. When vertical ground motions are used in the analysis, the lower-bound period should be established to also include the periods of the modes required to achieve 90% mass participation in the vertical direction. The above-noted period range applies to SFRSs listed in NBC Table 4.1.8.9. and similar ones. Appropriate period ranges for structures with a seismic isolation or supplemental energy dissipation system are given in NBC Clauses 4.1.8.19.(4)(c) and 4.1.8.21.(5)(c), respectively.

3. Selection of Ground Motions

This section provides guidance on the selection of ground motion time histories to cover the design spectrum specified in Section 2.1., over the period range, T_R, specified in Section 2.2. Suites of ground motion records should be selected in accordance with the criteria specified in Section 3.1. for each scenario-specific period range, T_{RS}, of the target spectrum (or spectra), as defined in Section 3.2. The minimum number of ground motion records for each suite is specified in Section 3.3. Guidelines for the selection of multi-component ground motion records are given in Sections 3.4. and 3.5.

3.1. Selection Criteria

Appropriate ground motions should be selected based on the tectonic regime, the magnitudes and distances that control the seismic hazard, and the local geotechnical conditions at the site. The response spectra of the selected motions should have spectral shapes that are similar to those of the target response spectrum (or spectra), as defined in Section 3.2.

Recorded ground motions are generally preferred; however, ground motions simulated using a seismological model may be used as an alternative if appropriate records are not available. If sufficient data exists, the ground motions for each suite should be selected from at least two distinct seismic events; where possible, no more than two ground motion records from the same earthquake event should be selected.

3.2. Horizontal-Component Target Spectrum, $S_T(T)$, and Scenario-Specific Period Ranges, T_{RS}

The target response spectrum (or spectra), $S_T(T)$, should be determined for the horizontal component of ground motion using one of the following methods (see Figure 4):

Method A: A single target response spectrum, $S_T(T)$, may be specified based on the design spectrum, as defined in Section 2.1., for the period range, T_R, as defined by Section 2.2. When Method A is used, the following considerations apply:

Suites of ground motion records should be selected to cover appropriate segments of the period range, T_R, considering the dominant earthquake magnitude–distance combinations revealed by the site-specific seismic hazard disaggregation. Each of the period segments constitutes a scenario-specific period range, T_{RS}. For locations where earthquakes from different tectonic environments (or sources) contribute to the hazard—as is the case in southwestern British Columbia where shallow crustal, subduction intraslab, and subduction interface earthquakes are expected—a minimum of one scenario-specific period range, T_{RS}, should be defined for each tectonic environment (or source) contributing to the hazard.

The scenario-specific period ranges, T_{RS}, may overlap each other, but together they should cover the period range, T_R, as defined in Section 2.2.

Method B: Two or more site-specific scenario target response spectra, $S_T(T)$, may be specified to cover the period range, T_R, as defined in Section 2.2. Each target spectrum is used to select and scale the ground motion records in lieu of the design spectrum, S(T). When Method B is used, the following considerations apply:

Suites of ground motion records should be selected for each site-specific scenario target spectrum, $S_T(T)$, considering earthquake magnitude–distance combinations and tectonic sources used to define the scenario target spectra. Each scenario target spectrum should cover a segment of the period range, T_R, as defined in Section 2.2., and each of the period segments constitutes a scenario-specific period range, T_{RS}. The T_{RS} ranges may overlap each other, but together they should cover the period range, T_R. The target spectra may be obtained from Method B1 or B2:

Method B1: Site-specific scenario target spectra, $S_T(T)$, are created for each dominant earthquake magnitude–distance combination and/or for each tectonic source that contributes to the hazard in the period range, T_R, as revealed by site-specific seismic hazard disaggregation. For locations where earthquakes from different tectonic sources contribute to the hazard—as is the case in southwestern British Columbia—a minimum of one scenario target spectrum is required for each source contributing to the hazard. The envelope of the scenario target spectra should be no less than the design spectrum, S(T), as specified in Section 2.1., over the period range, T_R, defined in Section 2.2.

Method B2: Alternatively, site-specific scenario target spectra, $S_T(T)$, are created for periods that correspond to those periods of the vibration modes that significantly contribute to the dynamic response of the building in the period range, T_R. Lengthening of the elastic periods due to anticipated inelastic response is accounted for when selecting the periods. For each period selected, a scenario target spectrum, $S_T(T)$, is created that matches or exceeds the design spectrum value at that period. When developing the scenario target spectrum, site-specific disaggregation should be performed to identify earthquake magnitude–distance combinations that dominate the hazard at each period considered. The scenario target spectra should be representative of one or more spectral shapes for the dominant earthquake magnitude–distance combinations revealed by the disaggregation; ground motion prediction equations (GMPEs) may be used to define the spectral shapes for specific scenarios; conditional mean spectra may be used as scenario target spectra. The envelope of the scenario target spectra should be no less than 75% of the design spectrum, S(T), as specified in Section 2.1., over the period range, T_R, defined in Section 2.2.

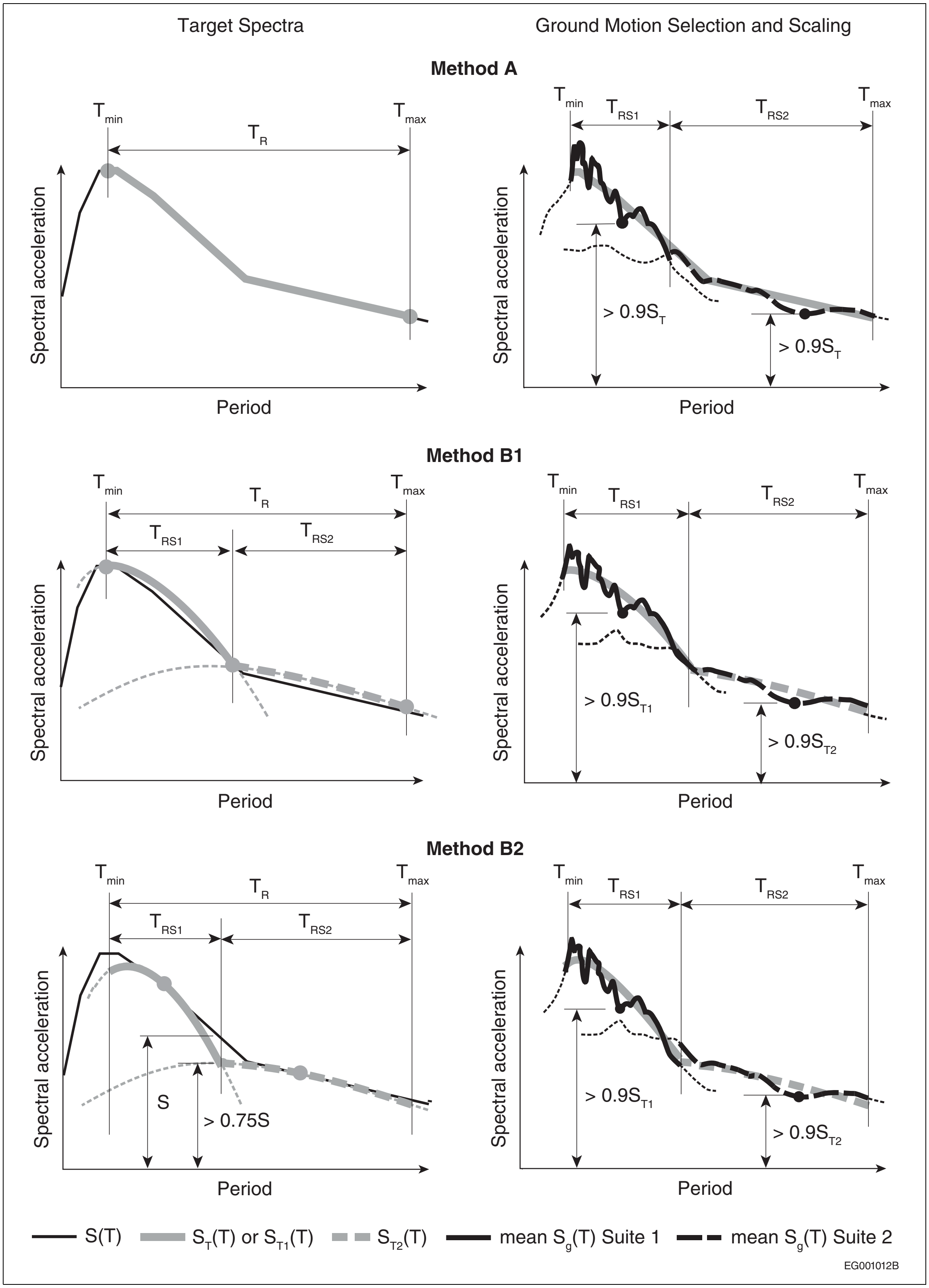


Figure 4

Definition of target spectrum (or spectra), $S_T(T)$, and scaling of suites of ground motion records over scenario-specific period ranges, T_{RS}, using Methods A, B1 and B2

3.3. Minimum Number of Ground Motions

When the target spectrum is defined using Method A in Section 3.2., the minimum number of ground motion records in each suite of each scenario-specific period range, T_{RS}, should be 5, but the total number of records in all suites should be not less than 11.[(1)] For locations where scenario-specific period ranges, T_{RS}, are required to represent the hazard from different tectonic sources, as specified in Section 3.2., a suite of not less than 11 records should be used for each source. When the scenario-specific target spectra are defined using Method B1 or B2 in Section 3.2., each scenario target spectrum should be matched using a suite of not less than 11 appropriate ground motion records. In both cases, using fewer than 11 records per suite may be permitted, provided the number of records for the suite is not less than 5, the number of records is approved by a peer review panel, and the total number of records considering all suites is not less than 11.

3.4. Components of Ground Motions

When analysis of the structure is performed independently in one horizontal direction, ground motion records should consist of appropriate single horizontal ground motion components.

When analysis of the structure is performed with orthogonal pairs of horizontal ground motion acceleration histories being applied simultaneously, ground motion records should consist of pairs of appropriate horizontal ground motion components. If possible, ground motion records should consist of pairs of orthogonal ground motion components recorded at the same station during the same earthquake event.

Where vertical ground motions are used in analysis, ground motion records should consist of appropriate vertical ground motion components. If possible, ground motion records should consist of horizontal and vertical ground motion components recorded at the same station during the same earthquake event.

3.5. Vertical-Component Target Spectrum

A vertical-component target spectrum may not be needed if historical ground motion records are used (see Section 3.4.). When needed, a vertical-component target spectrum may be developed using relationships between vertical and horizontal spectra that depend on site and soil conditions or be defined as a fraction of the horizontal-component target spectrum, which may be period-dependent. In the absence of site-specific information, a factor of 2/3 is often applied to the horizontal-component target spectrum to obtain the vertical-component target spectrum.

4. Scaling of Ground Motions

This section provides guidance for the scaling of ground motions to match the target spectrum (or spectra), $S_T(T)$, defined in Section 3.2.

Acceleration response spectra of the ground motion records are obtained as specified in Section 4.1. Records should be scaled in two stages, as specified in Section 4.2: in Stage 1, the records of each suite are scaled individually; in Stage 2, when necessary, all records in each suite are scaled by a second common factor. Figure 5 illustrates the scaling of three suites of 11 ground motion records selected for the three tectonic sources contributing to the hazard in southwestern British Columbia. Guidelines for scaling multi-component ground motion records are given in Sections 4.3. and 4.4.

4.1. Computation of Response Spectra for the Selected Ground Motions

Response spectral amplitudes of the selected ground motion records, $S_g(T)$, should be computed at period increments of no more than 0.02 s over the period range, T_R. No fewer than 20 such period values should span each scenario-specific period range, T_{RS}. The damping ratio used to compute the response spectra should match that of the target spectrum, $S_T(T)$, as defined in Section 3.2.

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(1) The number of records referred to in this Section is suitable to obtain the seismic demand corresponding to the design spectrum, but not its dispersion. A much larger number of records is needed if the dispersion is required.

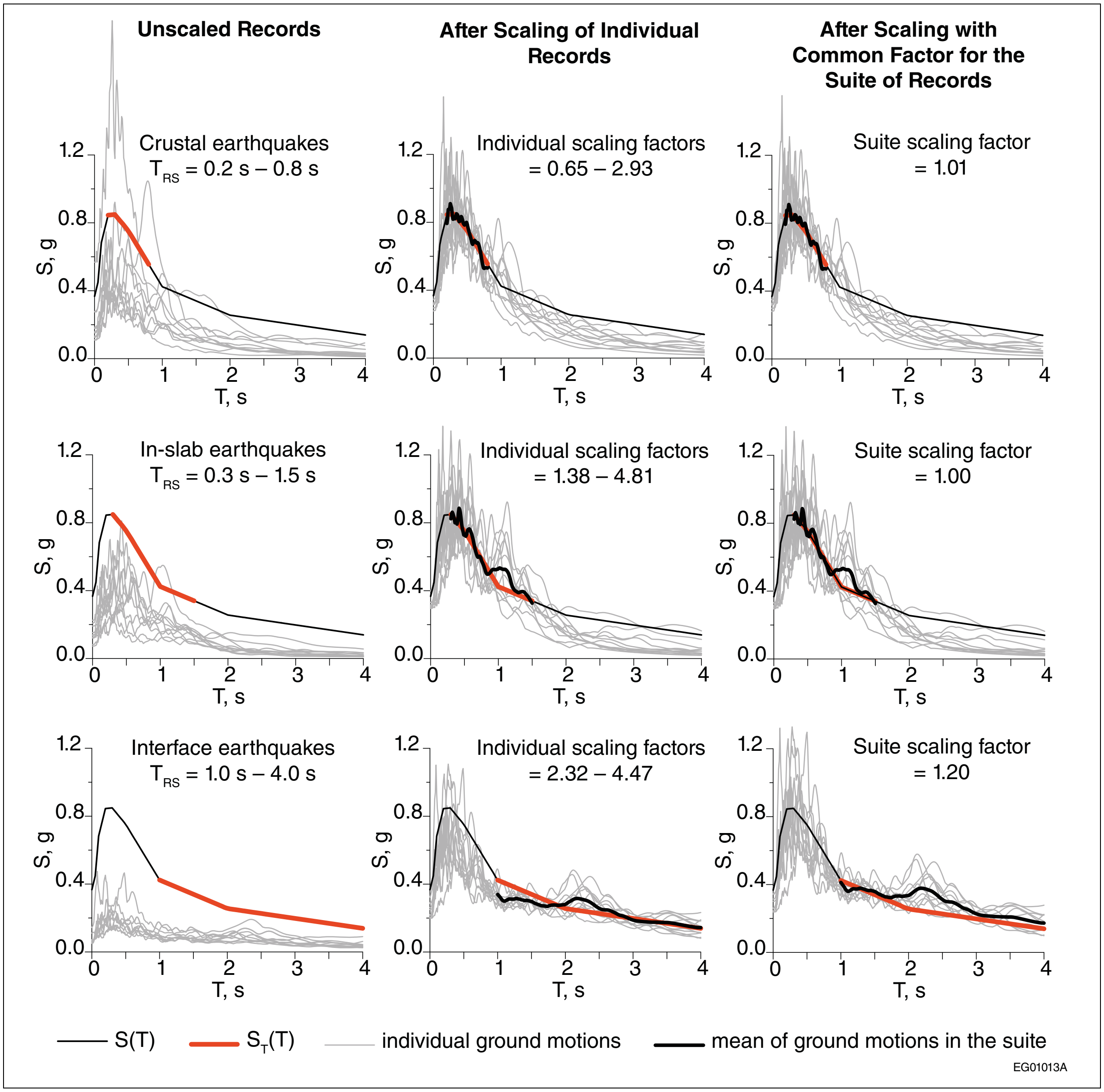

Figure 5
Selection and scaling of ground motion records for 0.2 s ≤ T_R ≤ 4.0 s for a Class C site in Vancouver using Method A

4.2. Scaling of Horizontal Ground Motion Components

Each ground motion should be scaled individually such that, on average, its response spectrum equals or exceeds the target response spectrum, $S_T(T)$, over the appropriate scenario-specific period range, T_{RS}, as defined in Section 3.2.

In addition, all records in each suite of time histories should be scaled by a second common factor such that the mean response spectrum of the suite, as defined in Section 3.3., does not fall more than 10% below the target spectrum, $S_T(T)$, in any period specified in Section 4.1., over the appropriate scenario-specific period range, T_{RS}, as defined in Section 3.2.

Caution should be exercised when excessively low or high scaling factors are required (e.g., less than 0.5 or larger than 4.0) as this may suggest that the ground motion is not compatible with the source mechanisms or seismic hazard level considered and that ground motion selection may need to be revised.

Frequency-domain and time-domain spectral matching techniques intended to match the target spectrum may be used with caution, while carefully evaluating the behaviour of the acceleration, velocity and displacement traces, including acceleration pulses, before and after spectral matching. When spectral matching techniques are used, all motion records should be scaled as described above except that the Stage 2 common scaling factor for the suite should be such that the mean response spectrum of the suite is not less than 110% of the target spectrum, $S_T(T)$, at any period value specified in Section 4.1., over the appropriate scenario-specific period range, T_{RS}, as defined in Section 3.2. The 110% factor need not be applied if each record of the suite is adjusted at fewer than 15 selected period values.

4.3. Scaling for Multidirectional Analysis

When performing multidirectional analysis, appropriate pairs of orthogonal horizontal ground motion components should be scaled with a single factor, according to the scaling procedure defined in Section 4.2. The factor should be such that the geometric mean of the spectra of the two horizontal components matches the target response spectrum.(2)

4.4. Scaling of Vertical Ground Motion Components

In the absence of a specific vertical-component target spectrum (Section 3.5.), the vertical component should be scaled by the same factor as the corresponding horizontal ground motion component(s) if multiple-component historical ground motion records are being used. When simulated ground motion components are used, they should be scaled to match the vertical-component target spectrum, as defined in Section 3.5., using the same scaling procedures as for the horizontal component (Section 4.2.).

5. Analysis Results

5.1. Unacceptable Response

Examples of unacceptable response are dynamic instability, non-convergent analysis, deformation demand on an element that significantly exceeds the valid range of modeling, and force demand on an element that exceeds the capacity of that element.

Unacceptable response should not be allowed, except under the following circumstances: for each suite, a single unacceptable response may be considered as an outlier if

- the suite includes a minimum of 11 ground motions;
- additional evaluations indicate that the predicted response is not indicative of unacceptable structural performance; and
- spectral matching techniques are not used.

When an outlier response is permitted, the results of the outlier analysis that produce the unacceptable response may be discarded and the design seismic demand as defined in Section 5.2. should be determined using the results of the remaining ground motion records.

5.2. Design Seismic Demand

When only one suite of ground motion records is used, the design seismic demand for a structural response parameter should be taken as the mean value of all ground motion records in the suite.

When using two or more suites of ground motion records that have been selected and scaled over scenario-specific period ranges, T_{RS}, as defined in Section 3.2., the design seismic demand of a structural response parameter should be taken as the largest of the mean values of each suite, provided that all suites contain a minimum of 11 ground motion records. If the number of ground motion records in any suite is less than 11, the design seismic demand of a structural response parameter should be taken as the mean of the n highest values of the response parameter among all records in all suites, where n is the average number of ground motions in all suites (e.g., n will be equal to 7 for the case where three suites of 6, 7 and 9 records, respectively, are used).

The above-noted design seismic demand values apply to SFRSs listed in NBC Table 4.1.8.9 and similar ones. Refer to NBC Sentences 4.1.8.20.(3) and 4.1.8.22.(1) for the requirements on obtaining

(2) The orientation of horizontal ground motion components should be varied, as described in NBC Article 4.1.8.8.

design seismic demand values for storey shears, storey forces, member forces and deflections for structures with seismic isolation and supplemental energy dissipation.

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Commentary K

Foundations

Commentary K

Foundations

Introduction

1. This Commentary provides guidance, compatible with sound engineering practice, for the design of foundations and temporary excavations in accordance with the provisions of NBC Section 4.2. NBC Subsection 4.1.3. requires the use of limit states design for the design of buildings and their structural components. This Commentary deals with this approach for the design of shallow and deep foundations. The material herein is intended as a first approximation dealing with routine problems of foundation design and construction. Neither this material nor the papers or texts to which it refers should substitute for the experience and judgment of a professional engineer competent in dealing with the complexities of foundation design practice.

2. This Commentary is divided into three principal parts: Temporary Excavations, Shallow Foundations, and Deep Foundations. Limit states design of temporary excavations has not yet been introduced and such excavations are to be designed according to the traditional allowable stress or global factor of safety procedures.

3. This Commentary does not deal specifically with the identification and classification of soils and rocks, with subsurface investigations, with swelling and shrinking clay, with frost action as related to foundations, with soil and hydrostatic pressures, or with retaining walls; these topics are included in the Canadian Foundation Engineering Manual (CFEM).[1]

Limit States Design

4. Limit states refer to those conditions of a structure in which the structure ceases to fulfil the function for which it was designed. The limit states are classified into two main groups:
   - ultimate limit states (ULS), and
   - serviceability limit states (SLS).

5. Ultimate limit states are primarily concerned with collapse mechanisms for the structure and, hence, safety. For foundation design, ultimate limit states consist of:
   - exceeding the load-carrying capacity of the foundation (i.e., ultimate bearing capacity),
   - sliding,
   - uplift,
   - large deformation of foundation, leading to an ultimate limit state being induced in the superstructure or building,
   - overturning, and
   - loss of overall stability.

6. Serviceability limit states consider mechanisms that restrict or constrain the intended use or occupancy of the structure. They are usually associated with movements that interrupt or hinder the purpose (i.e., serviceability) of the structure. For foundation design, serviceability limit states can be categorized as:
   - excessive movements (e.g., settlement, differential settlement, heave, lateral movement, and tilt or rotation), and
   - unacceptable vibrations.

7. The basic design equation for limit states design is:

$$\phi R_n \geq \Sigma \alpha_i S_{ni}$$

where ϕR_n is referred to as the factored geotechnical resistance. The resistance factor, ϕ, accounts for variability in the soil strengths and as-built dimensions, and variabilities introduced by inaccuracies in the calculation model. It also indirectly allows for ductile and catastrophic failures. The nominal resistance, R_n, is the engineer's best estimate of the ultimate resistance of the foundation. The value of R_n should allow, at least partially, for variabilities resulting from geotechnical uncertainties. It is based on the characteristic (nominal) strengths of the soil, nominal (specified) dimensions, and the normal calculation model.

8. The term S_{ni} is the nominal value of the forces on the foundation resulting from the ith load. These forces are obtained from the specified loads by structural analysis. The term α_i is the load factor for the ith load. It accounts for the variability in the load itself, approximations in the loading model given in the Code and variability introduced by the structural analysis.

9. The load factors and load combinations are as given in NBC Subsection 4.1.3.

10. The recommended resistance factors are given in Table K-1. The resistance factors in this Table have mainly been derived by direct calibration to traditional working (allowable) stress design. This means that the dimensions of foundations governed by bearing capacity should not be significantly different using limit states design procedure as compared to the working stress design procedure. The derivation of the resistance factor in Table K-1 is described in detail in Reference [2], where it is shown that the estimated reliability index, β, for shallow foundations using the resistance factors in Table K-1 ranges from 2.8 to 3.5, a range that is consistent with values commonly used for the design of the building structure. Figure K-1, which is taken from Reference [2], shows the relationship between global safety factor, resistance factor and reliability index, β, using statistical assumptions for variability in bearing resistance (coefficient of variation 0.3 and ratio of mean to nominal of 1.1) that is typical for shallow and deep foundations. The advantage of Figure K-1 is that β can be readily interpreted by geotechnical engineers who have considerable experience in using the traditional values of global safety factor. This can assist in bridging the gap, during the transitional stage, between the use of working stress and limit states concepts for geotechnical aspects of foundation design. Additional discussion on these aspects is provided in the CFEM.[1]

Table K-1
Resistance Factors for Shallow and Deep Foundations

Description	Resistance Factor
1. Shallow foundation	
(a) Vertical resistance by semi-empirical analysis using laboratory and in situ test data	0.5
(b) Sliding	
(i) based on friction (c = 0)	0.8
(ii) based on cohesion/adhesion (tan(f) = 0)	0.6
2. Deep foundation	
(a) Bearing resistance to axial load	
(i) semi-empirical analysis using laboratory and in situ test data	0.4
(ii) analysis using static loading test results	0.6
(iii) analysis using dynamic monitoring results	0.5
(iv) uplift resistance by semi-empirical analysis	0.3
(v) uplift resistance using loading test results	0.4
(b) Horizontal load resistance	0.5

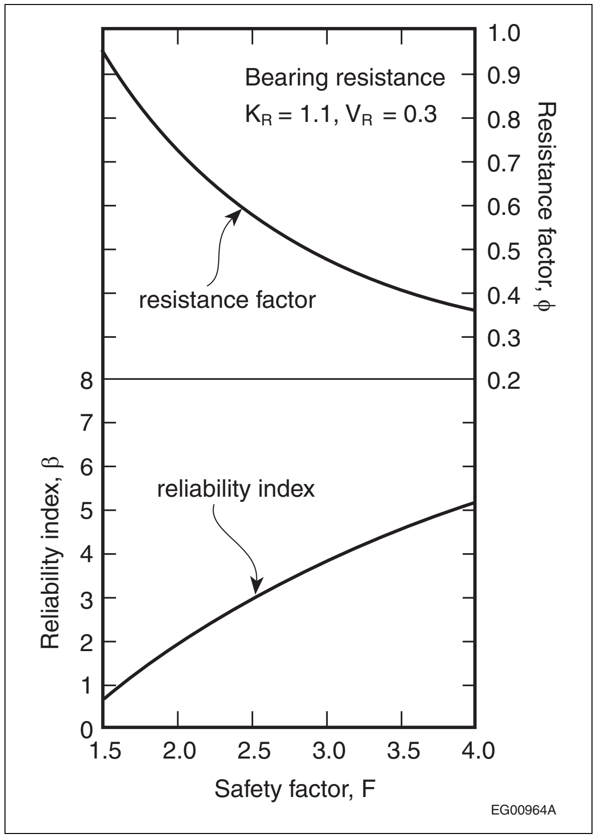

Figure K-1
Relation between safety factor, resistance factor and reliability index for bearing resistance, K_R = 1.1, V_R = 0.3

11. The selection of characteristic values of soil and rock properties, appropriate for the limit states investigated, shall be based on the results of laboratory and field tests and shall take account of the following:
(a) geological and other background information, such as data from previous projects,
(b) the variabilities of the property values,
(c) the extent of the zone of ground governing the behaviour of the geotechnical structure for the limit state considered,
(d) the influence of workmanship on artificially placed or improved soils,
(e) the effect of construction activities on the properties of in situ ground.

The selection of the characteristic value shall take into account the possible difference between the properties measured in the tests and the soil and rock properties governing the behaviour of the ground due to factors such as:
(a) the presence of fissures, which may play a different role in the test and in the geotechnical structure,
(b) the time effects, and
(c) the brittleness or ductility of the soil and rock tested.

12. In essence, the characteristic value corresponds to the geotechnical engineer's best estimate of the most appropriate likely value for geotechnical properties relevant for the limit states investigated. A cautious estimate of the mean value for the affected ground (zone of influence) is generally considered as a logical value to use as the characteristic value. Additional information and guidance on the selection of appropriate characteristic values are provided in the CFEM.[1]

13. In many cases, the variability of a mean value of a soil or rock property should be investigated, as well as the variability of an individual value resulting from a test. The extent of the zone of influence governing the behaviour of the ground for a limit state is usually much larger than the extent of the zone involved in a soil or rock test; consequently, the governing parameter is often a mean value over a certain surface or volume of the ground. An exception to this would be the presence of a weak layer, within the zone of affected ground, that would control the most likely failure mechanism (limit state). The appropriate characteristic value would be the mean or cautious estimate of the mean strength of the weak layer—not the mean strength of the affected volume of ground.

14. The governing zone of ground may also depend on the behaviour of the supported structure. For instance, when considering a bearing resistance for a building resting on several spread footings, where the building is unable to resist a local failure, the governing parameter would likely be the mean strength over each individual zone of ground under a footing. If instead the building is stiff and strong enough, the governing parameter may be the mean of these mean values over the entire zone or part of the zone of ground under the building.

Temporary Excavations

Unsupported Excavations

15. The safety and stability of unsupported excavations depends on the soil and groundwater conditions and on the depth and slope of the cut. In granular materials, slope failure will generally be fairly shallow; in clays, deep rotational failures involving not only the sides but also the base of the excavation are possible. The length of time that the cut will remain unsupported must also be considered.

Table K-2
Open Cut Excavation Guidelines(1)(2)

Category	Soil Type	Groundwater	Typical Failure Mode	Time to Failure	Remarks	Reference
A	Free-draining, granular, non-plastic silts	Below cut or controlled by advance dewatering	Shallow surface or slope wedge	Generally rapid	Rarely a problem if groundwater under control and slope angle does not exceed friction angle of soil. Unsaturated temporary steeper cuts rely on apparent cohesion and may slough with time; cuts steeper than 45° are not recommended; vertical cuts more than 1.2 m in depth should never be used.	[3]
B	Free-draining, granular, non-plastic silts	Cut below groundwater	Sloughing to flow	Rapid	Uniform fine soils may flow for considerable distances if pumping from within excavation is attempted. Slopes are controlled by hydraulic effects and may range from 1/3 or less to full value of friction angle.	[3]
C	Non-sensitive clays; plastic and cohesive silts	Saturated(3)	Rotational; plane of weakness or composite surface	Rapid or delayed depending on per cent of operational soil shear strength mobilized	Analytical methods are generally reliable for prediction of stability in soft to firm clays.	[3]
D	Sensitive clays	Saturated(3)	Rotational; retrogressive slides and as for Category C	As for Category C: little advance warning	Extreme caution required; once initial failure is provoked, retrogressive action may affect wide area; reliability of analytical prediction methods generally poor.	—

(1) Mixed soils such as glacial tills should be classified into Category A, B or C, depending on grain size, plasticity and permeability, and treated accordingly.

(2) The stability of an open cut slope, which is only marginally stable at the end of excavation, may be adversely affected by such factors as the nature and magnitude of crest loading, vibrations, rainfall, the length of time the cut remains open or disturbance of the soil in the vicinity of the toe of the slope.

(3) Excavations through alternate layers of cohesive and granular soils or excavations terminated within a cohesive soil underlain by granular strata require an investigation of groundwater conditions in each layer, and the factor of safety against excavation base heave or slope failure as a result of upward water pressure should be assessed.

16. Guidelines for the treatment of open cuts in broad soil categories are included in Table K-2. The selection of stable slope angles for Categories C and D requires that stability analyses be carried out. The selection of appropriate design shear strength parameters for such analyses requires a careful assessment of imposed shear stress levels, time effects, soil directional properties and uniformity, and should be carried out by a professional engineer qualified in this work. The influence of groundwater conditions within the slope, or piezometric levels at or below the toe of the proposed slope, should also be investigated, as the resisting shear strength along a potential failure surface may be greatly reduced by hydrostatic pressures. Additional information and guidance on the design of unsupported excavations are provided in the CFEM.[1]

Supported Excavations

17. Temporary shoring support of vertical excavation faces requires the assessment of a number of factors, including the length of time the excavation is to be supported, earth pressures, pressures from frost action and corrosion from aggressive soil or groundwater. The shoring wall elements may either be open, permitting full drainage, or closed, providing a barrier to groundwater flow, depending mainly on the soil permeability (hydraulic conductivity) and groundwater conditions. Closed systems are designed for soil and full groundwater pressures, whereas hydrostatic pressures are not included in open systems where seepage through the wall can take place. Additional information and guidance on the analysis and design of supported excavations are provided in the CFEM,[1] and in CSA S6, “Canadian Highway Bridge Design Code,” and its Commentary, CSA S6.1.

Earth Pressures

18. For flexible and semi-flexible shoring walls, which are commonly used to support the vertical faces of excavations and may have a variety of support conditions, no satisfactory general theoretical solutions for the prediction of earth pressures are available. The design earth pressure must take into account the method and sequence of construction and the tolerable deformation limits of the sides or faces of the excavation.

19. The yield of one part of a flexible wall throws pressure onto the more rigid parts. Hence, pressures in the vicinity of supports are higher than in unsupported areas, and the loads on individual supports vary, depending largely on the stiffness characteristics of the supports themselves and the construction technique.

20. The pressure envelopes, which represent the pressures that would normally be anticipated, can be represented in triangular, trapezoidal or rectangular form, and the applicable earth pressure coefficients will range between the active K_A* case and the earth pressure at rest K_O,** depending on permissible wall and soil movements.

21. **Non-cohesive (granular) soils.** As a first approximation, the guidelines in Table K-3 are suggested for essentially granular soils such as fills, sands, silts, sandy silts, gravelly sands, and gravels, or alternating layered conditions composed of such strata.

22. **Cohesive soils.** For cohesive soils, a distinction must be made between soft to firm clays and stiff to very stiff clays. The effects of clay sensitivity and the factor of safety against base heave must also be taken into account.

23. For stiff clay soils ($C_u > 50$ kPa) including silty clays, sandy clays and clayey silts, the guidelines in Table K-4 are suggested. Similarly, for soft (12 kPa < C_u < 25 kPa) to firm (25 kPa < C_u < 50 kPa) clays, reference should be made to Table K-5. (C_u = 1/2 unconfined compressive strength = undrained shear strength)

* $K_A = (1 - \sin\phi')/(1 + \sin\phi')$, where ϕ' = effective friction angle of soil and the ground surface is horizontal.

** K_O is frequently assumed to be equal to $1 - \sin\phi'$

Table K-3
Envelope of Earth Pressure for the Design of Temporary Supports for Granular Soils

Restraint	Design Total Pressure(1)	Envelope of Pressure Distribution(2)	Ability to Restrict Adjacent Soil Movements(3)
Cantilever	$1.0P_A$	Triangular	Generally very poor unless wall extremely stiff and embedded in dense soil
Braced	$1.2P_A$ to $1.3P_A$	Rectangular or trapezoidal	Generally poor where control of groundwater inadequate or where workmanship poor; can be moderate to good where these factors are properly controlled and bracing properly designed and tightly wedged or preloaded
Tied back	$1.1P_A$ to $1.4P_A$	Rectangular or trapezoidal	Generally good where high total pressures are used; movements usually less than for braced walls and dependent on degree of prestressing, workmanship and wall stiffness

(1) P_A = theoretical total active pressure = $0.5\rho gH^2 \times K_A$ where
ρ = total (bulk) density of soil (submerged if below groundwater), in kg/m³,
H = depth of cut, and
g = acceleration due to gravity, in m/s².
The value of 0.2 is suggested as a lower bound for K_A even in dense soils. Surcharge pressures, compaction-induced pressures, and hydrostatic water pressures should be added where appropriate.

(2) After increasing P_A by the appropriate multiplier, distribute total pressure over depth of cut as indicated in this column: triangular limits of trapezoid generally taken as 0.2H to 0.25H at top and bottom.

(3) Where greater control of adjacent ground movements is required, earth pressure should be computed using the at-rest K_o earth pressure coefficient with prestress in struts or tie-backs to the full design load. Additional measures would include choice of a stiff wall and close vertical spacing of struts or tie-backs.

Table K-4
Envelope of Earth Pressure for the Design of Temporary Supports for Stiff Cohesive Soils

Restraint	Design Total Pressure	Envelope of Pressure Distribution(1)	Ability to Restrict Adjacent Soil Movements
Cantilever	$1.0P_A$ but not less than $0.15\rho gH^2$(2)	Triangular	May be poor depending on length of cantilever, wall stiffness, embedment conditions and clay sensitivity(3)(4)
Braced or tied back	$0.15\rho gH^2$ to $0.4\rho gH^2$(5)	Rectangular or trapezoidal	Depends on soil strength, sensitivity, effective preloading or prestressing, and wall stiffness

(1) Surcharge pressures and compaction-induced pressures should be added where appropriate; hydrostatic pressures need not be included; total density of soil, ρ, is to be used in calculations.

(2) P_A may be computed using short-term strength, i.e., $P_A = \rho gH - 2C_u$, if the excavation is open for a limited period. Regardless of whether pressures are negative or zero, minimum positive pressures indicated should be used.

(3) Computed passive pressures below the base of the excavation should be reduced by 50% to account for unavoidable disturbance due to strain effects and stress release.

(4) The factor of safety against base heave in stiff over-consolidated clays, as a result of high locked-in lateral stresses, should also be investigated.

(5) Use higher range where clay is of high sensitivity. If the construction sequence or workmanship allow significant inward movement during any stage of excavation, pressures may build up to essentially fluid soil values in very sensitive clays. With good workmanship, clay pressures are similar to those given in Table K-2. Strength tests taken on intact samples of stiff clays that are jointed or fissured may overestimate the strength characteristics and thus lead to an underestimation of earth pressures. The effects of joints and fissures should be taken into account as appropriate to determine the operational strength of the soil mass.

Table K-5
Envelope of Earth Pressure for the Design of Temporary Supports for Soft to Firm Clays

Restraint	Design Total Pressure	Envelope of Pressure Distribution(1)	Ability to Restrict Adjacent Soil Movements(2)(3)
Cantilever	$1.0P_A$ but not less than $0.15\rho gH^2$(4)	Triangular	Very poor; this type of support generally to be avoided in soft, sensitive clays
Braced or tied back	$0.4\rho gH^2$ to $0.8\rho gH^2$(5)	Rectangular	Depends on clay shear strength and stability(6)

Table K-5 (Continued)

(1) Essentially fluid soil pressures in very sensitive clays may be realized as a result of unavoidable wall movements prior to insertion of restraint supports.
(2) Computed passive pressures below the base of the excavation should be reduced by at least 50% to account for unavoidable disturbance due to strain effects.
(3) Additional precautions in soft to firm sensitive clays would include (a) insertion of the top strut or anchor prior to excavation beyond 1.5 to 3 m depth, and (b) where the excavation area is of limited size, placing of a 150- to 300-mm-thick concrete mat at the base of the excavation, where practical, immediately on completion of excavation.
(4) P_A may be computed using short-term strength, i.e., $P_A = \rho gH - 2C_u$, if the excavation is open for a limited period. Regardless of whether pressures are negative or zero, minimum positive pressures indicated should be used.
(5) Higher range should be used where clay is of soft consistency, and lower range where clay is of firm consistency. This value may be conservative for non-homogeneous, non-sensitive sandy-silty cohesive soils of firm consistency. If stability number $N = (\rho gH + \text{surcharge})/C_u$ approaches 5 to 6, use the higher range. At this depth, base heave may also take place and suitable precautions should be taken.
(6) Design of a suitable shoring and bracing system in soft to firm clay conditions is not a routine matter, and the advice of a specialist should be obtained to establish earth pressures, to check overall stability and base heave, and to predict adjacent soil movements.

24. Earth pressure distributions calculated using nominal (unfactored) values of K_A or K_O and distributions based on Tables K-3, K-4 and K-5 represent nominal (specified) earth pressure distributions. In the calculation of lateral earth load for ULS conditions, these distributions are multiplied by appropriate load factors.

Movements Associated with Excavations

25. Movements associated with excavations are primarily related to construction technique and commonly consist of lateral yield of the soil and support system towards the excavation, with corresponding vertical movement adjacent to the excavation walls. Both lateral and vertical movements due to yield are generally of the same order of magnitude; however, if very flexible vertical wall elements are used, lateral movements can be grossly increased. Where construction technique is poor, erratic movements can also occur due to loss of ground or erosion behind the wall.

26. Movements due to yield of cantilever walls are related to the wall and soil stiffness. For most flexible or relatively flexible wall types, the lateral deformations will exceed the values required for the mobilization of active soil pressures. For most soils and particularly cohesive soils, there is a danger that a further buildup of lateral pressures beyond active values will take place as a result of loosening due to strain effects. An exception would be where lateral soil pressures of an at-rest magnitude or greater are used in design, and an appropriately stiff wall, such as large diameter cylinder piling, is provided and embedded in competent soil. Lateral earth pressure induced by compaction equipment should also be taken into account (CFEM[1] and CSA S6).

27. Movements due to yield in strutted excavations are, to a large extent, unavoidable, since they are controlled not by design assumptions but by construction details and procedures. Such movements develop in each excavation phase before the next level of struts is installed.

28. The yield movements of anchored walls are controlled to a larger extent by design methods more than is the case with strutted walls. The number of anchors and the vertical spacing of such anchors play a significant part in controlling the degree of lateral deformation. In normal practice, movements due to the yield of anchored diaphragms, sheeted or soldier pile walls are usually less than for strutted walls for the same depth of excavation.

29. For general guidance Table K-6 summarizes the approximate range of vertical and lateral movements to be expected. In certain cases, more favourable results may be achieved with proper design, good construction workmanship and careful field supervision, including monitoring the behaviour of the excavation. Additional information and guidance are provided in the CFEM.[1]

Table K-6
Vertical and Lateral Movements Associated with Excavation[1][2]

Restraint[3]	Wall Details	Granular Soils, % depth	Stiff Clay, % depth	Soft to Firm Clay,[4] % depth	Remarks
Cantilever	Conventional stiffness	Moderate to large	Moderate	May collapse	Movements related to wall, soil stiffness and embedment condition
Braced	Soldier piles or sheet piles	0.2 to 0.5	0.1 to 0.6[5]	1 to 2[5]	Struts installed as soon as support level reached and prestressed to 100% design load
	Rakers or struts loosely wedged	0.5 to 1.0	0.3 to 0.8	> 2	Poor workmanship would result in greater values
Tied back	Soldier piles or sheet piles	0.2 to 0.4	0.1 to 0.5	1 to 2	Prestressed to pressure between active and at-rest
	Concrete diaphragm walls	< 0.2	< 0.1 to 0.5	< 1 to 2	Prestressed as above, since wall stiffness and design earth pressures are normally greater, movements are generally less than for soldier piles or sheet piling; little data available

(1) Movements indicated apply directly behind wall; for granular soils and stiff clays, movements would feather out in approximately linear fashion over a horizontal distance of 1.0H to 1.5H, where H is the depth of excavation. For soft to firm clays, and assuming average workmanship, this distance increases to 2.0H to 2.5H, and with poor workmanship to greater than 3H.
(2) If groundwater is not properly controlled in granular strata, movements may be much larger than indicated, and loss of ground could also result.
(3) Experience indicates that movements are reduced by using close vertical spacing between strut or tie-back levels and by careful attention to prestress details.
(4) If the factor of safety against base heave for soft to firm clays is low, large deformations will result.
(5) Upper range of movements usually applies for highly sensitive clays in either stiff or soft to firm category.

Underpinning

30. Structures adjacent to excavations frequently need to be supported. The need for underpinning depends on the location of the structure, the details of its foundation support, its sensitivity to settlement and lateral deformations, the cost of underpinning or provision of extra excavation face support and other precautions, and the cost of repairs or the consequences if the structure is not underpinned.

31. The geometry of zones within which support for adjacent structures is usually considered necessary, as a result of adjacent excavation through soil, is shown in Figure K-2. Where adjacent structures are founded on bedrock and excavation is through rock, less underpinning and more face support should be considered.

32. The general order of magnitude of movements as a result of excavation with various support methods in different soil conditions has been summarized in Table K-6. This Table may also be used to assist in judging the necessity for underpinning. Additional information and guidance can be obtained from the CFEM.[1]

Factors to Be Considered with Soil and Rock (Ground) Tie-Back Anchors

33. Anchors are usually inclined downwards, transmitting the vertical component of the anchor force into the anchored vertical member. This force should be considered in design, together with the weight of the vertical member itself.

34. Forces that resist downward movement due to the inclined anchor load are skin friction and the reaction at the base of the vertical member. When soldier piles are used, vertical forces are concentrated in the piles. Only minimal friction, if any, can be mobilized. Such vertical forces are supported at the base of the pile. The vertical and horizontal base capacity of the pile should be checked; otherwise, unacceptable vertical and horizontal deformation may take place.

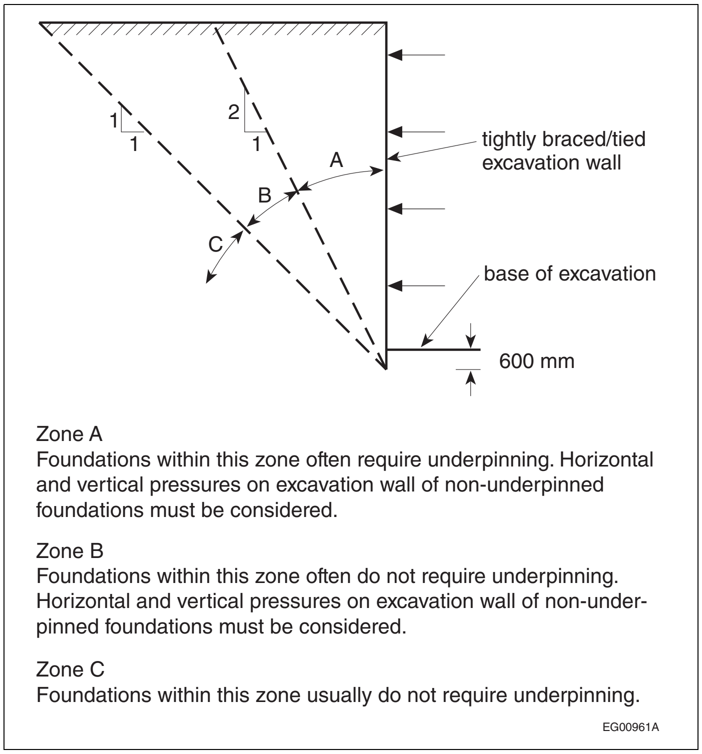

Figure K-2
Requirements for underpinning

35. Settlement of vertical members produces some reduction in anchor loads, with a consequent tendency for outward displacement of the supported face. Vertical and horizontal movements at the top and bottom of the excavation should be monitored at regular intervals throughout the course of the work.

36. The performance of soil and rock anchors depends not only on minor variations in soil and groundwater conditions but also on construction techniques and details. Consequently, the prediction of anchor capacity by theoretical calculations may not be reliable. Anchorage capacities should be established by load test, taking into account the load deformation and "creep" properties of the ground, and each anchor should be proofloaded during construction.

37. The overall stability of a ground anchorage system should be checked by analyzing the stability of the block of ground lying between the wall and the anchorages. In general, the anchors should be extended beyond a 1:1 line drawn from the base of the excavation, and no allowance for any load-carrying support should be assumed within this line.

Design and Installation of Members

38. Members such as walers, struts, soldier piles and sheeting should be sized in accordance with the structural requirements of NBC Part 4.

39. The depth of penetration of the vertical wall member should be at least 1.5 times the depth required for moment equilibrium about the lowest strut.

40. For driven soldier piles, the maximum horizontal force on the flange of the soldier pile below the bottom of the excavation may be taken as 1.5 times the values computed for the width of the flange, providing that the pile spacing is not less than five times the flange width.

41. For piles placed in a concrete base, the diameter of the concrete filled hole may be used in place of the flange width as discussed in the preceding paragraph.

42. The selection of material and sizes of timber planks or lagging should conform with good practice, and the lagging should be of good quality hardwood. Lagging is installed by hand after a depth of about a metre is excavated. The maximum depth made each time before a section of lagging is placed depends on the soil characteristics. Soft clay and cohesionless soils must be planked in short depths to reduce the amount of soil moving into the excavation. The depth of excavation below any lagging boards that have not yet been placed should not exceed 1.2 m. Lagging should be tightly backfilled or wedged against the soil.

43. To minimize the possibility of erratic loss of ground in local areas when excavating sands and silts below original groundwater, straw packing, burlap or in extreme conditions, grouting should be used behind the lagging as it is installed.

44. The design of all members including struts, walers, sheet piling, walls and soldier piles should be checked for several stages of partial excavation when the wall is assumed to be continuous over the strut immediately above the excavation level and supported some distance below the excavation level by the available passive resistance. This condition could produce the maximum loading in struts and walers.

45. Where excessive stresses or loads would result from interim construction conditions using regular construction procedures, trenching techniques can be employed to advantage.

46. The design of members should also be checked for the scenario when portions of the building within the excavated area are completed and lower struts are removed. Consideration must be given to the possible increase in loading on the upper struts remaining in place; also the span between that portion of the building that has been completed and the lowest strut then in place should be considered in relation to flexural stresses.

Control of Groundwater in Excavations

47. Good practice requires that the following conditions be fulfilled when dewatering excavations:
- (a) A dewatering method should be chosen that will not only assure the stability of the sides and bottom of the excavation but will also mitigate damage to adjacent structures, such as by settlement.
- (b) The lowered water table should be kept constantly under full control, and fluctuations liable to cause instability of the excavation must be avoided.
- (c) Effective filters must be provided where necessary to prevent loss of ground.
- (d) Adequate pumping and standby pumping capacity must be provided.
- (e) Pumped water must be discharged in a manner that will not interfere with the excavation or cause pollution.
- (f) For most soils, the groundwater table during construction must be maintained at least 600 to 1 500 mm below the bottom of the excavation so as to achieve dry working conditions. The groundwater table should be maintained at a somewhat lower level for silts than for sands in order to prevent traffic from pumping water to the surface and making the bottom of the excavation wet or "spongy."
- (g) Adequate monitoring of groundwater levels by piezometers or by observation standpipes should be maintained.
- (h) Where low permeability strata are underlain by pervious water-bearing layers, depending on the depth of excavation and the hydrostatic head in the pervious strata, it may be necessary to lower the head in the pervious stratum in advance of excavation, to prevent a "blow" or excessive disturbance of the base as a result of upward hydrostatic pressure.
- (i) Pumping from sumps or ditches inside the excavation is normally carried out where dense, low permeability soils, such as certain glacial tills or cohesive soils, are present or where the excavation is in bedrock. This method is not recommended for excavation in semi-pervious or pervious soils, such as silts or fine sands, because it often leads to extensive sloughing of the excavation sides and disturbance of the bottom.

Shallow Foundations

General

48. A shallow foundation means a foundation unit that derives its support from the soil or rock close to the lowest part of the building that it supports. The depth of the bearing area below the adjacent ground is usually governed by the requirement to provide adequate protection against climatic or frost effects; vertical loads on the sides of the foundation due to adhesion or friction are normally neglected.

Limit States Design Procedure for Shallow Foundations

49. The limit states to be considered are as discussed in Paragraphs 4 to 6.

50. When designing a spread (shallow) foundation, one of the following design methods shall be used:
(1) The Direct Method, in which separate analyses are carried out for each limit state using calculation models recommended in the CFEM[1] and appropriate load factors and resistance factors described in this Commentary (refer to Table K-1). In the case of serviceability limit states related to settlement, the settlement under the service loads is determined in accordance with the methods given in the CFEM,[1] using characteristic (nominal) soil properties. In the case of ultimate limit states related to bearing capacity, the foundation forces due to factored loads (including wind or earthquake) are compared with the factored geotechnical resistance (i.e., nominal ultimate resistance multiplied by the resistance factors given in Table K-1).
(2) The Empirical Method, in which geotechnical resistances/pressures estimated empirically in the CFEM[1] are compared to the pressures due to the specified loads. The serviceability limit pressures in the CFEM[1] are generally based on a maximum settlement of approximately 25 mm. The Empirical Method is convenient for the initial design of foundations of buildings as well as for the final design of most ordinary buildings. The foundations of tall buildings or towers, special buildings sensitive to movements or buildings on sensitive ground, however, should be evaluated using the appropriate direct procedures described in item (1).

51. The following limit states terms should be used for expressing recommended geotechnical criteria for the design of the building structure, including its foundations:
Bearing pressure for settlement means the bearing pressure beyond which the specified serviceability criteria are no longer satisfied. This is also referred to as serviceability limit pressure.
Factored bearing resistance means the calculated ultimate bearing resistance, obtained using characteristic soil parameters, multiplied by the appropriate recommended resistance factor (refer to Table K-1).
Factored sliding resistance means the calculated ultimate sliding resistance, obtained using characteristic soil parameters, multiplied by the appropriate recommended resistance factor (refer to Table K-1).
Factored pull-out resistance (i.e., against uplift) means the calculated ultimate pull-out (uplift) resistance, obtained using characteristic soil parameters, multiplied by the recommended resistance factor.

Ultimate Bearing Capacity and Settlement (Serviceability)

52. The design of a foundation unit requires that both ultimate bearing capacity (ULS condition) and settlement (SLS condition) be checked. In many circumstances, settlement (serviceability considerations) governs the design. Distress from differential settlement is usually evidenced by cracking and distortion of doors and window frames. Bearing capacity (ultimate limit states) failures are rare, except perhaps during construction, where shallow temporary footings are frequently used with falsework.

53. The ultimate bearing capacity of cohesive and non-cohesive soils can be determined with reasonable reliability by assuming that the strength parameters for the bearing soil are accurately known within the depth of influence of the footing. The ultimate bearing capacity of shallow foundations can be calculated using classical bearing capacity formulae or semi-empirical correlations with the results of in situ testing such as the standard penetration test (N values) or the cone penetration test. Correlations with laboratory tests such as uniaxial compression tests are frequently used to estimate bearing capacity and ultimate anchor bond resistance for bedrock. Characteristic (nominal ultimate) soil and rock strength properties are used in the classical bearing capacity formulae. The prediction of ultimate bearing capacity is multiplied by an appropriate resistance factor to provide factored bearing resistance. Additional information and guidance are provided in the CFEM[1] and in CSA S6.

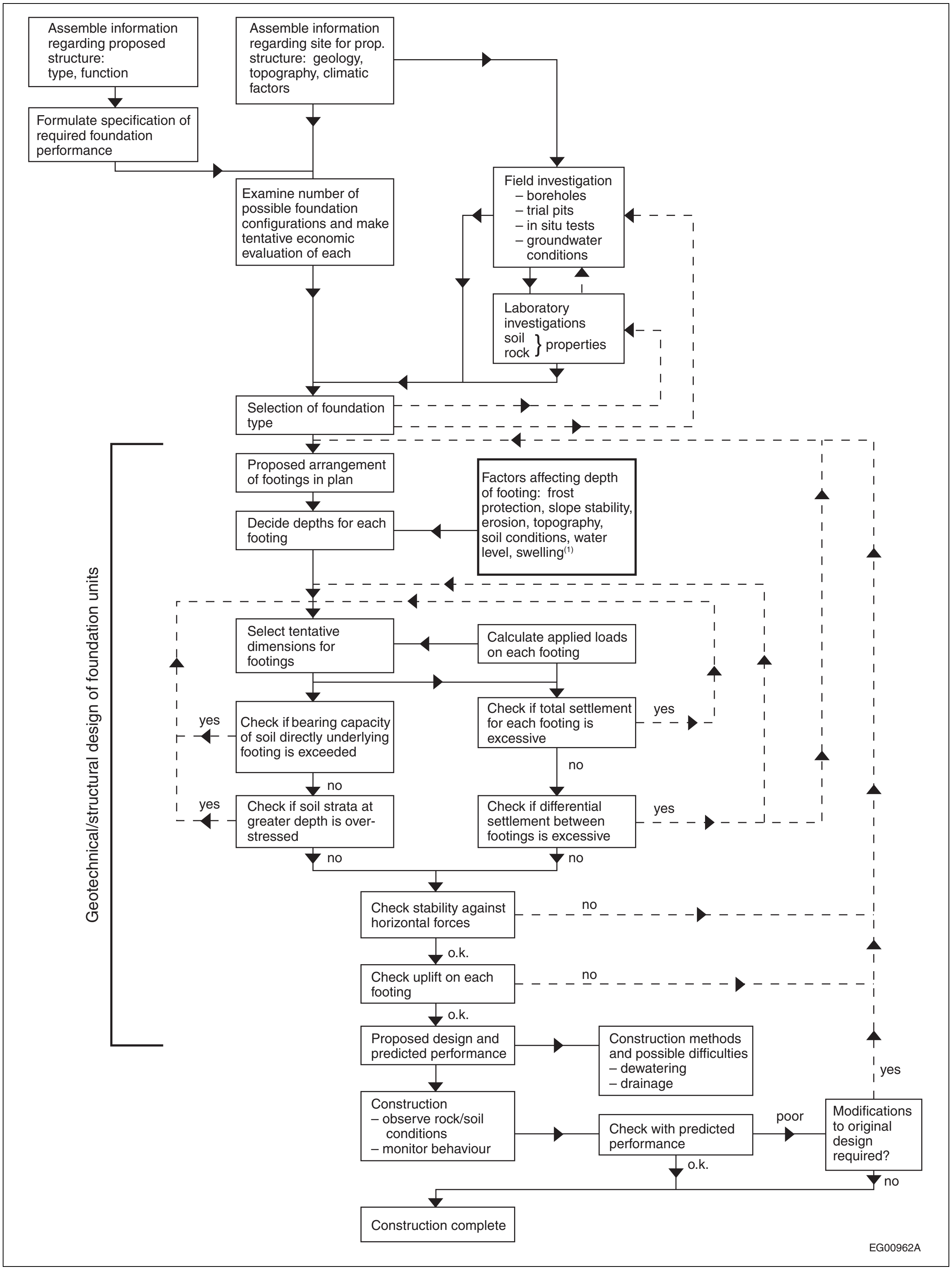

Figure K-3
Flow diagram for the design of shallow foundations

Note to Figure K-3:

(1) These factors frequently govern foundation design.

54. **Cohesive soil.** The settlement of a structure on cohesive soil is affected by a number of complicating factors usually requiring experience and judgment to assess. The most important of these is an estimate of the preconsolidation pressure, that is, the maximum past consolidation pressure on the in situ soil. Because of the various uncertainties, errors of a factor of 2 are not uncommon in the calculation of settlement. Cohesive soils also display significant time-dependent (post-construction) settlement. Elastic theory, with appropriate modifications, can predict settlement with reasonable accuracy. Many other theoretical and empirical methods are available to predict settlement of shallow foundations (CFEM,[1] and CSA S6 and its Commentary, CSA S6.1).

55. **Non-cohesive soil.** The settlement of a structure on non-cohesive soil is normally estimated by empirical and theoretical methods. Settlement in granular (non-cohesive) soils generally occurs quite rapidly, often during the construction period. Post-construction settlement is usually negligible.

56. Post-construction settlement can occur for a considerable period after construction, even after a period of successful performance of the structure, as a result of vibrations or changes in the groundwater conditions, whether natural or man-made, due to earthquake or blasting, flooding or groundwater lowering.

Basis for the Design of Shallow Foundations

57. In limit states design, the relevant limit states are identified and through the design process shown in Figure K-3, it is verified that no limit state is exceeded. The design process may be simplified in many cases as experience will often show which type of limit state will govern the design and the other limit states are checked to ensure that they are not exceeded. Guidance is provided only for footings supporting vertical loads.

Estimates of Bearing Pressure for Settlement (Serviceability)

58. In traditional working (allowable) stress design, allowable bearing pressure was frequently controlled by settlement (serviceability) considerations. Normally the design pressures were such that total settlement would not exceed 25 mm and differential settlement would not exceed 19 mm. Preliminary design can usually be performed on the basis of the ground description and condition. However, final design should confirm these preliminary estimates following normal analytical (calculation) procedures and in keeping with good geotechnical practice.

59. Estimated values of presumed serviceability limit pressures (bearing pressure for settlement) are given in Tables K-7 to K-9 for bedrock and soil materials. Experience has shown that these values generally limit total and differential settlement of footings to 25 mm and 19 mm, respectively. If serviceability limit states correspond to different settlement criteria, these values would not be appropriate. The values given in these Tables should be treated as first approximations only and should be considered as maximum permissible values in the absence of additional information and data.

Table K-7
Estimates of Serviceability Limit Pressures on Rock

Rock Type	Rock Conditions(1)	Serviceability Limit Pressure,(2) MPa	Remarks
(a) Massive igneous and metamorphic rocks in sound condition; granite, diorite, basalt and gneiss	Discontinuities (joints, minor cracks) at wide spacing (> 1 m)	10	—
	Discontinuities at moderate spacing (300 mm to 1 m)	2 to 5	
(b) Foliated metamorphic rocks in sound condition: slate and schist	(i) Discontinuities at wide spacing (> 1 m)	3	Foliations approximately horizontal.
	(ii) Discontinuities at moderate spacing (300 mm to 1 m)	< 1	Foliations approximately horizontal.
	(iii) Foliations tilted to the horizontal	—	Potential sliding along foliations. Potential lack of support adjacent to cuts on excavations. See Reference [4].
(c) Sedimentary rocks in sound condition: cemented shale or siltstone, sandstone, limestone, dolomite and heavily cemented conglomerate	Discontinuities at wide spacing (> 1 m)	1 to 4	Strata approximately horizontal.
		—	Potential solution cavities in limestone, dolomite. Variability in cementation of conglomerates. See (b)(iii).
(d) Compaction shale and other argillaceous rocks in sound condition	Discontinuities at wide spacing (> 1 m)	0.5 to 1	Strata approximately horizontal.
		—	Argillaceous shales are subject to some swell on release of stress. All shales tend to soften on exposure to water and certain shales swell markedly.
(e) All closely jointed rocks including thinly bedded limestones and shales	Discontinuities at spacing less than 300 mm apart, random joint or crack patterns	—	Can only be assessed by detailed investigations and examination in situ, including loading tests if necessary.
(f) Heavily shattered or weathered rocks	—	—	See (e).

(1) Spacing of discontinuities is critical to the bearing pressure allowable on a rock mass. Discontinuities, such as joints or cracks, are considered widely spaced if greater than 1 m apart and moderately spaced when greater than 300 mm. The thickness or width of such discontinuities is presumed to be less than 5 mm (or less than 25 mm if completely filled with soil or rock debris). Where such conditions do not exist, Type (e) or (f) must be assumed.

(2) Values of bearing pressures given above, except for (f), are based on the assumptions that the foundations are close to the rock surface but carried down to unweathered rock with adequate frost protection and that the foundation is greater than 300 mm wide.

Table K-8
Estimates of Serviceability Limit Pressure on Non-Cohesive Granular Soils

Soil Type and Conditions(1)	Serviceability Limit Pressure,(2) kPa	Potential Problems(3)	Remarks
(a) Dense well-graded sands, dense sand and gravel	400 to 600	Density of sands containing large sizes or gravels is frequently overestimated when inferred from standard or cone penetration tests only. See Reference [5].	For general reference, see References [1] and [6].
(b) Compact well-graded sands, compact sand and gravel	200 to 400		
(c) Loose well-graded sand, loose sand and gravel	100 to 200	Potential settlement when subject to shock or vibrations. See (f).	
(d) Dense uniform sands	300 to 400	Density usually better defined by standard or cone penetration tests, as compared to (a) to (c). Considerable caution required in interpretation of test data.	See References [7] to [9].
(e) Compact uniform sands	100 to 300		

Table K-8 (Continued)

Soil Type and Conditions(1)	Serviceability Limit Pressure,(2) kPa	Potential Problems(3)	Remarks
(f) Loose uniform sands	< 100	Even where very low bearing pressures are used, settlement can occur due to submergence, vibrations from blasting machine operation or earthquake.	See Reference [10].
(g) Very loose uniform sands, silts	—	Subject to possible liquefaction. Should never be used for support of foundations.	—

(1) Density condition of the soil is assumed to be established in conformance with good geotechnical practice.
(2) Values are based on the assumptions that the foundation width, B, is not less than 1 m and that the groundwater level will never be higher than a depth B below the base of the foundation. When the groundwater level is, or could be, higher than such depth, the values listed should be divided by a factor of 2. Total and differential footing settlements are expected not to exceed 25 mm and 19 mm respectively.
(3) Long-term settlement of foundations on compact to dense non-cohesive deposits is normally modest, provided such deposits are not underlain by compressible cohesive deposits at depth.

Table K-9
Estimates of Serviceability Limit Pressure on Cohesive Soils (for sensitive clays, see Table K-10)

Soil Type and Conditions(1)	Serviceability Limit Pressure,(2) kPa	Applicability for Support of Shallow Foundations(2)	Settlement(2)(3)
(a) Very stiff to hard clay, heterogeneous clayey deposits or mixed deposits such as till	300 to 600	Good	Normally estimated on the basis of investigations, sampling and laboratory test data. For general reference, see References [1] and [11] to [13].
(b) Stiff clays	100 to 200	Fair to good	
(c) Firm clays	50 to 100	Poor, except for minor structures little affected by distortion	
(d) Soft clays	0 to 50	Very poor, not recommended	
(e) Very soft clays	—	Not permitted	

(1) Strength of cohesive soils is assumed to be established in conformance with good geotechnical practice.
(2) Cohesive soils are susceptible to long-term consolidation settlement. For Types (b) to (d) inclusive, such long-term (post-construction) settlement often governs the design. In the case of Type (a) soils, heave can take place with excavation and consequent relief of stress.
(3) Total and differential footing settlements are expected not to exceed 25 mm and 19 mm, respectively.

60. Table K-10 identifies problematic ground conditions where presumed values cannot be estimated without detailed investigations and analysis.

Table K-10
Problem Soils, Rocks or Conditions(1)

Type or Condition	Examples	References
Organic soils	Muskeg terrain: estuarine organic silts and clays	[14]
Normally consolidated clays	Lacustrine deposits and varved glacio-lacustrine deposits in Manitoba, Northern Ontario, Northern Quebec	[15]
Sensitive clays	Marine clay deposits in St. Lawrence River Valley, Eastern Ontario, Quebec	[16][17][18]
Swelling/shrinking clays	Clay-rich deposits in Alberta, Saskatchewan, Manitoba	[19]
Metastable soils	British Columbia loess	[20]
Expansive shales	Western Canada – Bearpaw and Cretaceous deposits Eastern Canada – weathering of sulphide minerals accelerated by oxidizing bacteria	[21][22]
Permafrost	Northern Canada, Arctic	[23][24]

(1) No ultimate bearing pressure or serviceability limit pressure can be presumed without detailed investigations.

Total and Differential Settlements

61. The total and differential settlements and relative rotations for foundations shall be estimated to ensure that these do not lead to the occurrence of an ultimate limit state or a serviceability limit state, such as unacceptable cracking or jamming of doors, in the supported structure. This requires attention and interaction between the geotechnical and the structural engineers.

62. The maximum acceptable relative rotations for open frames, infilled frames and loadbearing or continuous brick walls are likely to range from about 1/2 000 to about 1/300 to prevent the occurrence of a serviceability limit state in the structure, and about 1.5 to 2 times these values for long-term movements over many years because of creep of building materials. A maximum relative rotation of 1/500 for short-term movements and 1/300 for long-term movements is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150.

63. For normal structures with isolated foundations, the maximum acceptable differential settlement is about 20 mm between adjacent columns. On sand, the differential settlement of foundations is unlikely to exceed 75% of the maximum settlement and the maximum total settlement should not exceed about 25 mm. For a raft foundation, the maximum total settlement may be increased to 50 mm. The maximum allowable total and differential settlement may be increased in the case of foundations on clay soils provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the building, with tilt, etc. The above guides concerning limiting settlements apply to simple routine buildings. They should not be applied to buildings that are out of the ordinary or for which the loading intensity is markedly non-uniform.

64. Differential settlements calculated without taking account of the stiffness of the structure tend to be overpredictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements.

65. Differential settlement caused by variability of the ground should be taken into consideration unless it is prevented by the stiffness of the structure. For spread foundations on natural ground, the magnitude of the differential settlement may typically be up to 10 mm, but it does not usually exceed 50% of the calculated total settlement.

66. Calculation models for settlement analyses are given in the CFEM.[1] It is important to keep in mind that differential settlement of isolated footings will always occur because of the natural variability of soils.

67. In situations where calculation models are not available or are considered to be unnecessary, limit states may be avoided by the use of prescriptive measures. Prescriptive measures may be used, for example, to ensure durability against frost action and chemical or biological attack. These measures involve conventional and generally conservative details in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures.

Frost Penetration

68. The best assessment of frost penetration in a particular locality is local experience. In the absence of local experience, however, daily air temperature measurements can be used to estimate the combined effects of both depth and duration of freezing. The cumulative total of the difference between daily mean air temperatures and the freezing point is known as the "freezing index," which is expressed in Celsius degree-days. Freezing index values for a large number of weather stations in Canada are available from Environment and Climate Change Canada at ec.enviroinfo.ec@canada.ca. Figure K-4 shows the average freezing index for regions of Canada for the period 1978–2007 and Figure K-5 shows the 50-year-return-period freezing index for regions of Canada for the period 1958–2007. The contour lines in Figure K-5 were estimated by fitting the 2-parameter Weibull distribution to the annual average freezing index values for each location (see the example in Reference [25]). Information on how the freezing index can be used to estimate depth of frost penetration is given in CFEM[1] and References [26] to [29].

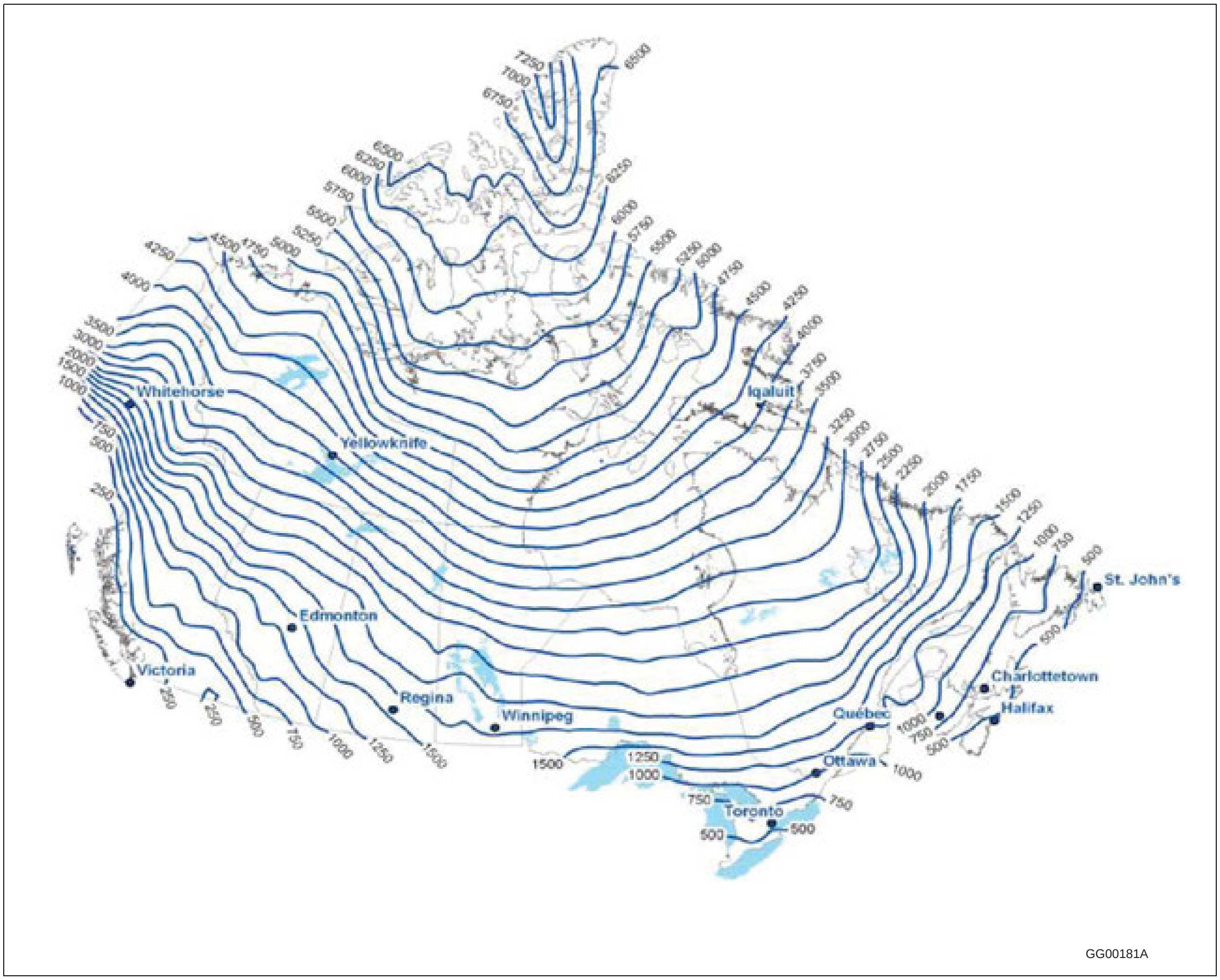

Figure K-4
Annual average freezing index (Celsius degree-days) based on the period 1978–2007

Insulated Shallow Foundations

69. Lightweight plastic insulation has been used to reduce the loss of ground heat and thereby reduce the depth of frost penetration. Insulation should be used for this purpose only after careful examination of the pertinent conditions and with a thorough understanding of its effect on the temperature at the soil–foundation interface.[29] Insulation is of particular benefit in the design of unheated buildings such as warehouses, garages and refrigerated buildings. It is also used to restrict the depth of frost penetration beneath artificial ice surfaces.

70. Insulation with relatively high compressive strengths can be obtained, so that slabs of these materials can be placed directly below the bearing surfaces of foundations. Substantial economic advantages may accrue where such designs are used, because foundations can be located closer to the ground surface, thereby reducing the costs of providing granular fill to replace frost-susceptible soil.[29] Design guidance is also given in the CFEM.[1]

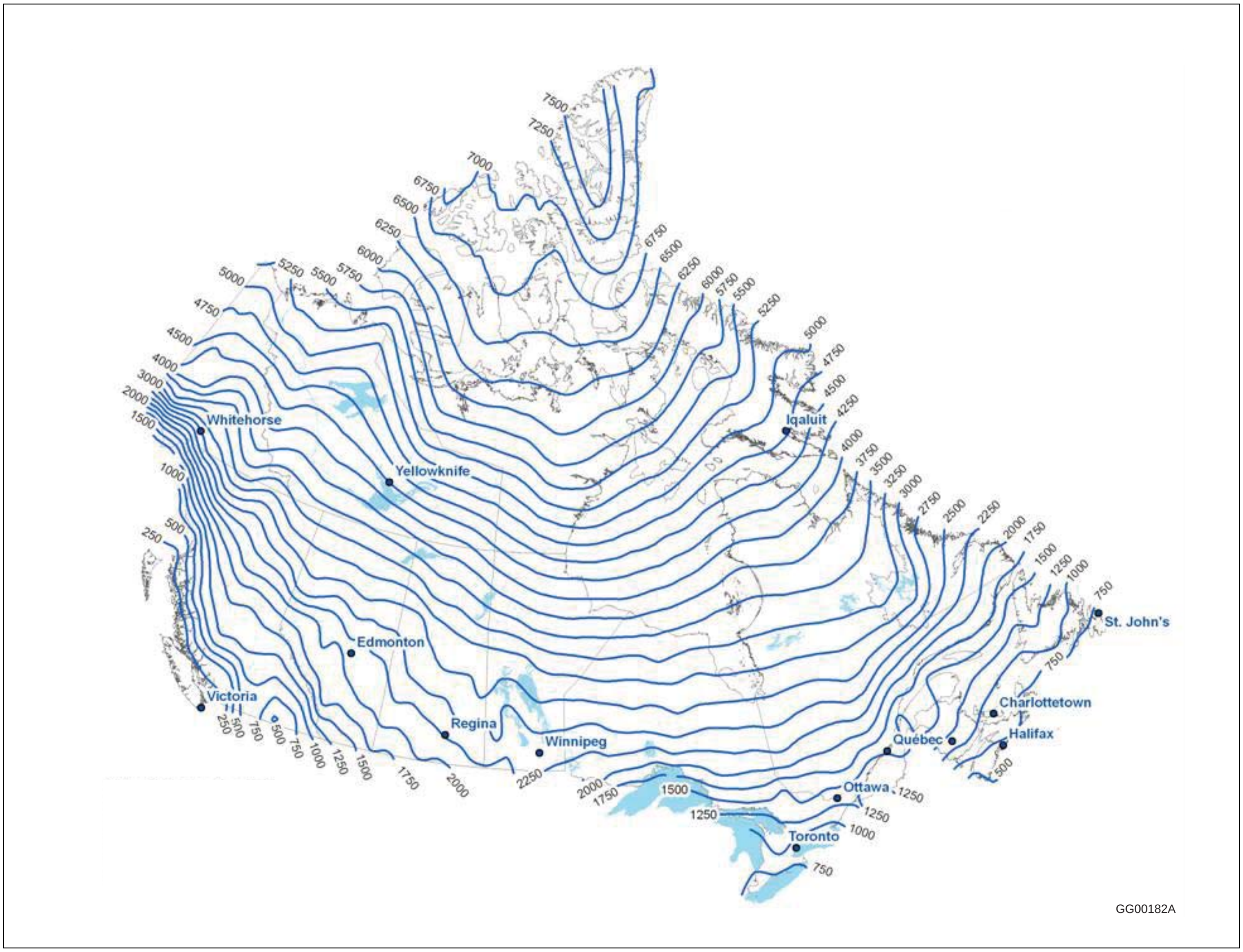

Figure K-5
50-year-return-period freezing index (Celsius degree-days) based on the period 1958–2007

Deep Foundations

General

71. A deep foundation is a foundation unit that provides support for a building by transferring loads either by end-bearing to a soil or rock at considerable depth below the building, or by adhesion or friction, or both, in the soil or rock in which it is placed. Piles are the most common type of deep foundation.

72. Piles can be pre-manufactured or cast-in-place; they can be driven, jacked, jetted, screwed, bored, drilled or excavated. They can be of wood, concrete, steel or a combination thereof. (Drilled shafts of diameter greater than about 750 mm are frequently referred to as "caissons" in Canada.)

Limit States Design Procedure for Piles

73. The limit states to be considered are as discussed in Paragraphs 4 to 6.

74. The ultimate limit states for pile foundations should also consider structural failure of the pile in compression, tension, bending, buckling or shear.

75. The design of pile foundations shall be based on one of the following methods:

(a) empirical or analytical calculation models as recommended in the CFEM[1] in which separate analyses are carried out for each limit state with appropriate values for the loads and characteristic soil parameters and appropriate resistance factors as described in this Commentary (Table K-1), or
(b) the results of load tests which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience.

76. The load factors and load combinations are as outlined in NBC Section 4.1.

77. The characteristic values for the geotechnical parameters are selected as discussed in Paragraphs 11 to 14.

Geotechnical Requirements of Deep Foundations

78. Loads that may be applied to a deep foundation depend not only on the properties of the foundation as a structural unit (e.g., the shaft strength of a drilled shaft determined on the basis of CSA A23.3, "Design of Concrete Structures"), but also on the properties of the foundation soil (or rock) and of the soil/foundation system (e.g., pile capacity as a function of soil strength, settlement of a drilled shaft as a function of contact pressure). Thus, the designer must distinguish the structural from the geotechnical capacity of a deep foundation unit or system, analyze each very carefully and define the application of loads that may be safely carried, both from a structural and a geotechnical point of view. In many applications, geotechnical considerations limit the permissible loads to levels well below those that might be arrived at on the basis of structural considerations alone. An exception to this possibly occurs when the pile is founded on strong bedrock (CSA S6) or other ground considered to be unyielding.

79. Geotechnical criteria for assessing the permissible loads on a deep foundation are determined on the basis of site investigations and geotechnical analyses. However, in most cases, the quality of a deep foundation is highly dependent on construction technique, equipment and workmanship. Such parameters cannot be quantified or reliably taken into account in normal design procedures. Consequently, as implied in NBC Subsection 4.2.7., the design capacity/performance of deep foundations should be confirmed on the basis of in situ load tests on actual foundation units.

80. Criteria relating to structurally permissible loads are defined in the design sections of the NBC applicable to the structural materials used in the deep foundation unit. However, the standards referenced in the NBC were written mainly for the purpose of designing elements and assemblies in the superstructure. A structural designer involved in the design of deep foundations must recognize that installation and quality control conditions below grade differ from those above grade; the permissible loads determined by the usual structural design methods may have to be reduced, sometimes to a marked degree, to account for these differences. Permissible loads can only be selected on the basis of close cooperation and interaction between the geotechnical and structural engineers for the project.

81. In this section of the Commentary, suggested values of permissible service loads are given for several kinds of foundation units. These values are listed solely to provide a first approximation of the probable loads which, under routine conditions, might be safely applied to a given kind of unit. In each case, both geotechnical and structural evaluations and analyses are mandatory. However, as discussed above, because construction procedures often have a dominant influence on the load/deformation behaviour of the deep foundation, the choice of a permissible service load is always subject to judgment and experience and to the provision that appropriate review be carried out as specified in NBC Article 4.2.2.3. Review must be considered an integral part of the design process.

82. Deep foundations that are placed on rock or on a dense basal deposit, such as till or hard clay, are bored, drilled or excavated and cast-in-place, and are commonly referred to as drilled shafts. In this case, the area of end-bearing contact is known and, provided this area and the character of the foundation stratum can be defined by inspection, the serviceability performance of the deep foundation can be evaluated on the basis of the serviceability limit pressure of the foundation stratum. (Refer to Tables K-7, K-8 and K-9 on shallow foundations.) Additional information and guidance for design is provided in the CFEM,[1] and in CSA S6 and its Commentary, CSA S6.1.

83. **Rock sockets.** Frequently, cast-in-place foundations are socketed into rock, either to obtain higher end-bearing capacity at depth or to transfer load to the rock by adhesion or bond along the walls of the socket. Adhesion is highly dependent on the rock type and on the socket wall condition after drilling. Characteristic (nominal) values used for adhesion in sound rock lying below weathered or shattered rock range from 0.7 MPa to 2.0 MPa; however, much lower values have been observed in practice, where the construction methods used have produced a poor contact area. Careful inspection of all rock sockets prior to concreting is essential. Socketing may also be employed to provide base fixity and resistance to horizontal movement.[30][31]

84. Deep foundations may also be driven to rock or into dense basal deposits. In this case, which includes H-piles, pipe piles driven closed-end or precast concrete piles, the exact area of contact with the foundation stratum, the depth of penetration into it or the quality of the foundation stratum are largely unknown. Consequently, the load capacity of such driven deep foundations should be determined on the basis of observations during driving, load tests and local experience. (Refer to Table K-11.)

Table K-11
Load Capacities of Driven Piles

Pile Type	Load Capacity	Recommendations	References
(a) End-bearing on rock, dense till or other similar materials	High to very high, but dictated by driving conditions, conditions of basal deposits, pile types and stiffness	Ultimate pile capacity usually high but load/deformation can only be assessed by load test (ASTM D 1143/D 1143M, Method A).	—
(b) Piles driven into dense sand, sand and gravel	See (a).	See (a).	[32][33]
(c) Piles driven into loose to compact sand, sand and gravel	Medium to high, part point resistance, part skin friction	First approximation to load capacity, use skin friction(1) = 50 ± 25 kPa. Define by load test (ASTM D 1143/D 1143M, Method A).	[32][33][34][35]
(d) Piles driven into compact to dense silts	Medium, but “relaxation” effects must be checked	See (c). Essential to define by load test.	[36]
(e) Piles driven into cohesive soils	Low to medium, susceptible to long-term settlement	First approximation, use skin friction.(1) Soft cohesive soil, 0 – 30 kPa. Firm to stiff cohesive soil, 30 – 60 kPa. Define by load test (ASTM D 1143/D 1143M, Method B).	[37][38]

(1) The skin friction values refer to characteristic (nominal) values.

Piles in Granular Soils

85. Piles that are driven into granular soils derive their load-carrying capacity from both point resistance and shaft friction. The relative contributions of point resistance and shaft friction to the load-carrying resistance (capacity) of the pile depend essentially on the density of the soil and on the characteristics of the pile.

86. It is commonly assumed that pile driving in granular soils increases the density of the deposit. Because of this, piles in granular soils should be driven to the maximum depth possible, without causing pile damage, in order to obtain the maximum working (service) load on the pile. However, in some granular soils, such as fine sands or cohesionless silts, the pile resistance (capacity) may decrease after driving. This effect is known as “relaxation.” In contrast, in some coarse sands or other coarse grained deposits, the load-carrying capacity of piles may increase after driving. Neither of these effects can be assessed quantitatively, except on the basis of redriving and load testing.

87. **Compacted concrete piles.** Compacted or rammed concrete piles in granular soils derive their load-carrying capacity mainly from the densification of the soil around the base. The capacity/resistance of such piles is, therefore, entirely dependent on the construction technique and can only be assessed on the basis of load tests and detailed local experience.

Piles in Cohesive Soils

88. The load-carrying capacity of piles driven into cohesive materials is governed by the adhesion between the pile and the soil and, to a much lesser extent than in granular soils, by the point resistance. This is particularly true for soft to firm clays.

89. The adhesion is not always equal to the undrained shear strength of the soil because, in some circumstances, the effect of pile driving markedly changes the character of the soil. In soft sensitive clays, complete remoulding of the soil may occur on driving. This effect diminishes with time following driving, as the soil adjacent to the pile consolidates. In some cases, soil strength has not returned to the original undisturbed value even after a considerable period of time.[39]

90. Because of the slow rate of regain of strength in certain cohesive soils, load testing should sometimes be delayed until several weeks have elapsed after driving.

91. In stiff to very stiff cohesive soils, evidence indicates that, in driving, a gap is formed between the pile and soil; this gap is not always fully closed with time, thus minimizing the adhesion to the pile relative to the high shear strength of the soil. For this reason, an approximate limit of 60 kPa has been suggested for the adhesion value, even for stiff clays (Table K-11).

92. **Drilled shafts in cohesive soils.** Except for shafts drilled through stiff or very stiff cohesive deposits, the major portion of drilled shaft capacity/resistance is derived from the hard or dense stratum at the base. For a first approximation of service loads, Tables K-7 and K-8 may be used. For a more detailed assessment of bored piles, see Reference [40].

Spacing and Arrangement of Piles and Drilled Shafts

93. The following should be considered during the spacing and arrangement of piles and drilled shafts:
- (a) the overlap of stresses between units, which influences total load-carrying capacity and settlement,
- (b) overstressing of weaker zones at depth, and
- (c) installation difficulties, particularly the effects on adjacent piles or drilled shafts.

94. In most cases the spacing, D, between the centres of driven piles of average diameter, d, should not be less than 2.5d.

Settlement and Group Effects in Piles

95. In practice, piles are frequently used in groups; however, most of the published literature deals with the behaviour of single piles. Leonards[41] states that, "there is no consistent relationship between the settlement of a single pile and the settlement of the pile group at the same load per pile. Therefore, selecting a design load on the basis of the load at a given gross or net deflection, or at a given fraction of the ultimate pile capacity, is equivalent to accepting an unknown factor of safety with respect to satisfactory performance of the foundation." This statement is certainly valid for all piled foundations where the piles derive their support from skin friction, or from combined skin friction and end-bearing; however, group effects may be less critical where piles derive all of their support or the major portion of it from end-bearing on a relatively incompressible stratum. An example of such support is where piles are driven through weak deposits to end-bearing on rock. For this case, the engineer normally relies on some means of assessing the dynamic resistance during pile driving complemented by load tests to define the deformation characteristics of the piles under load.

96. In contrast to true end-bearing pile foundations, where the load/deformation characteristics of individual piles are significant, the use of friction pile foundations is generally governed by considerations of group action and, for cohesive soils, long-term consolidation settlement. The actual capacity and load/deformation characteristics of individual piles are not significant in this case. The purpose of friction piles in the upper part of a deep deposit of cohesive soils or of granular soils (or silts) is to reduce the intensity of pressure acting at ground level and to shift the zone of maximum stress to the lower levels, where less settlement will result.

97. In the case of an individual pile or where the building is narrow in relation to the depth of piles, the zone of pressure increase is spread over a large area in comparison with the width of the foundation. In contrast, where the building is wide, friction piles spread the load out very little, and the effect of the pile foundation on the soil is practically the same as that of a raft foundation without piles. In this case, the resistance of the group of piles in the foundation bears no relation to the resistance of an individual pile by itself; the settlement of the foundation is, therefore, governed by the character of the subsoil, not by the load capacity of the piled foundation.

Load Tests on Deep Foundations

98. Use of load tests. As previously indicated, load testing of piles is the most precise method of determining load-carrying capacity for ULS and SLS conditions. Depending upon the type and size of the foundation, load tests may be performed at different stages during design and construction.

99. Load tests during design. The best method of designing a pile foundation is to perform pile driving and loading tests. The number of tests, the type of pile tested, the methods of driving or of installation and of test loading should be selected by the professional engineer responsible for the design. The following points should be considered:

(a) The test program should be carried out by a person competent in this field of work.
(b) Adequate geotechnical information should be obtained at the test location.
(c) The piles, the equipment used for driving or other method of installation, and the procedure should be those intended to be used in the construction of the foundation.
(d) As a minimum, the head of a pile should be instrumented to record the total pile and soil deformation. Where possible, deformation measurements should also be made at the tip of the pile and at intermediate points to allow for a separate evaluation of point resistance and skin friction.
(e) The driving process should be observed in detail and, wherever possible, stress levels in the pile assessed (e.g., by means of the wave equation method of analysis).
(f) The piles should be loaded to at least twice the proposed service load and preferably to failure.

100. Load tests during construction. Load tests should be performed on representative deep foundation units at early stages of construction. The purpose of such tests is to ascertain that the loads obtained by design are appropriate and that the installation procedure is satisfactory.

101. The selection of the test piles should be made by the professional engineer responsible for the design on the basis of observed driving behaviour or installation features.

102. Load tests for control. Where full advantage is to be taken of NBC Clause 4.2.4.1.(1)(c) and NBC Sentence 4.2.7.2.(2), a sufficient number of load tests must be carried out on representative units to ascertain the range of the pile performance under load. Load tests for control should be performed on one out of each group of 250 units, or portion thereof, of the same type and performance criteria. Load tests should also be performed on one out of each group of units where driving records or other observations indicate that the soil conditions differ significantly from those prevailing at the site. Selection of the deep foundation units to be load tested is the responsibility of the design engineer.

Installation and Structural Requirements of Deep Foundations

103. In most cases, the load-carrying capacity/resistance of a deep foundation unit is governed by geotechnical considerations. The capacity of a deep foundation unit determined from structural considerations represents the maximum axial load that could theoretically be carried; however, this load is generally less than could be applied to a comparable unit used in the superstructure of a building because

(a) the actual placing of deep foundations frequently deviates from the position and alignment assumed in design,
(b) once in place, deep foundation units often can neither be inspected nor repaired, and
(c) the placement of concrete in cast-in-place deep foundations frequently cannot be done with the same degree of control as in structural columns.

104. In Tables K-12 to K-14 guidelines are given to assist in determining a reasonable axial service load for deep foundation units under common conditions. These Tables are not a substitute for structural analysis and design, but only provide a conservative guide for routine situations that a designer may encounter, where a unit may be considered as a short column and where axial load governs the design.

Table K-12
Guidelines for Driven Piles

Type of Pile	Normal Size Range, mm	Typical Pile Load, kN	Structural Considerations	Installation Considerations	Notes
(a) Timber	180 to 250 (tip)	180 to 450	Must be checked in accordance with NBC Subsection 4.3.1.	Cannot be inspected. Susceptible to damage during hard driving. Tip reinforcement recommended where driven to end-bearing stratum.	Preservative treatment normally required. (CAN/CSA-O80 Series)
(b) Steel sections (H, WF)	200 to 350	350 to 1 800	Must be checked in accordance with NBC Subsections 4.3.3. and 4.3.4. In pipe piles, concrete strength does not normally contribute to pile capacity unless the pile is driven to end-bearing stratum.	May be damaged during driving but load-carrying capacity not necessarily reduced.	Tip points often required for hard driving. Average thickness of flange or web, t ≥ 10 mm. Projection of flange ≤ 14t.
(c) Pipe sections	200 to 600 (diam)	350 to 1 800		Suitable for inspection after driving. Concrete quality highly dependent on placement method.	Normally driven closed-end. Tip reinforcement required or drive to be visible when driven open-end. Pipe thickness > 5 mm, but 10 mm recommended.
(d) Precast concrete sections	200 to 300	350 to 1 000	End bearing: capacity must be checked in accordance with NBC Subsection 4.3.3. Normally f'_c > 27.5 MPa. The capacity of friction piles is normally governed by both installation method and geotechnical considerations; average compressive stress under load rarely exceeds 10 MPa.	Cannot be inspected. Careful selection and driving method required to prevent damage.	Refer to ACI 543R. Possible tensile stresses in concrete during "soft" driving. High compressive stresses in concrete during "hard" driving. Tip reinforcement usually essential.
	300 to 900	900 to 2 500			

Table K-13
Guidelines for Compacted Expanded-Base Piles

Type of Pile	Normal Size Range, mm	Typical Load, kN	Structural Considerations	Installation Considerations	Notes
(a) Rammed shaft	350 to 600	450 to 1 350	Concrete quality is highly dependent on technique.	Cannot be inspected. Contamination of concrete. 'Necking' of shafts. Possible damage by adjacent piles.	Load frequently determined on the basis of energy required to expel measured volumes of concrete at base. Highly dependent on judgment and experience. Possible heave of all piles must be continuously monitored.
(b) Steel pipe shaft, concrete filled	300 to 500	450 to 1 550	Where the pipe wall thickness < 5 mm, the structural contribution of the pipe should be disregarded.	Less subject to damage than (a). Shaft can be inspected prior to filling.	See (a).

Table K-14
Guidelines for Drilled Shafts

Type of Shaft	Normal Size Range of Shaft, mm diam	Typical Load, kN	Structural Considerations	Installation Considerations	Notes
(a) Uncased plain concrete	300 to 700	250 to 450	Good concrete quality is not always possible.	Where shaft diameter < 700 mm, cannot normally be inspected.	Not recommended for normal application where caving can occur.
(b) Uncased; reinforced or plain concrete; under-reamed or straight	750 to 1 500	450 to 45 000	Generally good concrete quality is possible with 35 MPa > f'_c > 20 MPa. Can normally be designed in accordance with NBC Subsection 4.3.3. (CSA A23.3).	Can be inspected. Where temporary casing is used to retain wet, caving soil, high slump concrete may be required. Precautions should be taken to prevent contamination of concrete.	Usually under-reamed to provide belled base. Bell sides typically at 2(V) to 1(H). Often not under-reamed where bearing on sound rock.
(c) Cased; permanent steel pipe lining	450 to 1 500	450 to 45 000	See (b). Must be checked as composite unit in accordance with NBC Subsections 4.3.3. (CSA A23.3) and 4.3.4.	Can be inspected.	Usually not under-reamed. Generally socketed where taken to rock. Design for complete load transfer through socket. Essential to seat liner on rock bearing surface. Drive shoe usually fitted to pipe liner.

105. The flexural capacity and ductility of piles should be considered when, under certain soil conditions, the soil either does not provide lateral support or could cause lateral loads to be applied to the piles.

106. Frequently, savings can be had by using piles with a higher capacity/resistance or different techniques. Higher performance requirements should only be used in conditions where they can be justified as suitable and when quality can be ensured through an adequate program of inspection and load tests.

Driven Piles

107. This type of deep foundation unit may suffer structural damage while being driven. Determination of capacity/resistance is generally made by comparing driving resistance (blows per 30 cm) with the energy or size of hammer blow and relating these values to previous experience or to the behaviour of similar piles subjected to static load tests. For this purpose, observations of pile driving must include:
(a) pile length and weight,
(b) hammer type (e.g., drop, diesel, ram weight),
(c) hammer energy applied,
(d) type and thickness of packing,
(e) blows per 30 cm and elastic rebound of pile, or
(f) acceleration and stress at head of pile.

108. The assessment of pile stresses during driving by the theory of wave propagation or by the “wave equation” method of pile analysis is useful. By assigning appropriate elastic properties to such parameters as the pile/cushion system and the pile/soil system, the penetration per blow and pile stresses for a given hammer energy can be computed; however, these results and the extrapolation of the penetration per blow to a definition of ultimate pile capacity are, at best, only approximations. The “wave equation” method, in common with all empirical dynamic pile formulae, calls for the exercise of judgment and experience. No method, in itself, can provide definitive values either for driving criteria or load/deformation characteristics of a driven pile. Pile load tests are essential to confirm the driving criteria used and to assess load/deformation performance.

109. **Damage to driven piles.** Piles may be damaged by attempting to drive to an excessively small “set” per blow or to an excessively large number of blows at high resistance. This is known as “overdriving.” The driving set should be established so as to achieve a reasonable performance under load without incurring the risk of serious damage. Driving stresses depend upon the hammer, blows, size and type of pile, length of pile, cushion material and soil conditions. These factors must be examined for each situation and acceptable "set" criteria determined on the basis of previous experience and load testing.

110. Piles may also be damaged by driving through obstructions, such as boulders or fill material, or by sloping rock surfaces, which may deflect the pile or create high local stresses leading to serious deformation or breakage.

111. Excessive bend or sweep may be experienced when driving long piles (30 m or more). A discussion of permissible bending of piles is given in Reference [42].

112. The use of steel reinforcing tips is strongly recommended whenever ends may be damaged. Tip reinforcement may also reduce damage incurred through overdriving.

113. Movement of adjacent piles during driving. Where a group of piles is to be placed through silt or clay, measures shall be taken to indicate any movement of each pile during the installation of adjacent piles. Horizontal and vertical movement should be recorded.

114. Piles that have sustained vertical movement should generally be redriven. Piles that have sustained horizontal displacement must be investigated for structural damage.

115. Jetting or pre-excavation. When jetting, predrilling or other pre-excavation methods are used during pile installation, the pile tip should be driven below the depth of pre-excavation to the required bearing stratum. Care must be taken to avoid jetting, pre-driving or pre-excavating to a depth or in a manner that will affect the design capacity/resistance of previously placed piles. This is discussed in detail in ACI 543R, "Recommendations for Design, Manufacture, and Installation of Concrete Piles."

Cast-in-place Deep Foundations

116. Cast-in-place deep units can be divided into two main categories: compacted expanded base piles (Table K-13) and drilled shafts (Table K-14).

117. The placement of the materials forming such units is crucial. It is difficult, if not impossible, to ensure the same level of quality in placing concrete in such units as in a building superstructure. Careful attention must be given to the methods of installation, concrete mix proportions and placement methods, and to the degree of inspection possible. The performance requirements of such units should be adjusted accordingly, in keeping with sound design, engineering experience and judgment.

118. Concrete cast in place. The placing of concrete in pipe piles, expanded base pile shafts and in drilled shafts can be classified in two categories:

(1) Concrete placed in dry conditions should be placed by guided free fall, bucket or chute. Segregation may occur if concrete is allowed to fall through a reinforcing cage or similar obstruction. Concrete of more than 100 mm slump placed by free fall of 5 m or more in unreinforced or lightly reinforced shafts generally receives adequate compaction and does not usually require vibration. Placement by tremie methods is preferable in most cases and is necessary when a considerable inflow of groundwater is present or when there is standing water in the hole.

(2) Concrete placed under water should be placed through a tremie pipe or by pump in such a way as to eliminate any contamination, washing or dilution of the concrete by the water. It should have a 150 to 200 mm slump and vibration should not be applied. (Refer to CSA A23.1, "Concrete Materials and Methods of Concrete Construction.")

119. Reinforcing steel for cast-in-place units. Reinforcing steel is generally placed pre-assembled as in a cage. During placement, the steel may be subjected to severe handling and placement stresses and to impact. Placement cannot be made with as high a degree of accuracy as in a superstructure, nor can it be easily checked.

120. For the design of cast-in-place foundations, the provisions of CSA A23.3 should therefore be amended in the following respects:

(a) Reinforcing steel assemblies should be designed and constructed so as to withstand all handling and placing stresses without deformation, which would impair the structural performance of the unit.

(b) Weldable steel should be employed, in most cases, to permit construction of rigid and strong assemblies.

(c) The clear distance between longitudinal bars should not be less than 75 mm.
(d) Ties or spirals may be welded to the longitudinal bars. Welding should be in accordance with CSA W59, "Welded Steel Construction (Metal Arc Welding)." Welded spirals or ties should be of wire not less than 7.0 mm in diam, with a pitch not more than 300 mm and with not less than 75 mm clear space between ties or spirals.
(e) The possibility of misplacing the reinforcing bars should be allowed for in the design, and reasonable tolerances established for field performance: e.g., within ±75 mm of correct bar location in plan, within ±150 mm of correct bar location in elevation.
(f) Generally, longitudinal steel should be uniformly distributed around the cross-section, as an assembly may become twisted during placement.

Location and Alignment

121. The exact location of each deep foundation unit should be staked in advance and checked immediately prior to the installation of each unit. After completion of the installation, the location of each unit should be checked against design location and permissible deviation as indicated on the design documents.

122. As required in NBC Article 4.2.7.3., permissible deviations from the design location shall be determined by design analysis. In practice, piles and shafts can usually be positioned within a tolerance of 80 mm; for practical reasons smaller tolerances should not be specified.

123. As required in NBC Article 4.2.7.4., where a deep foundation unit is wrongly located, the condition of the foundation shall be assessed by the person responsible for the design and the necessary changes made.

124. During and after installation of any deep foundation unit, its alignment should be checked against the design alignment and the permissible deviation as indicated on the design documents.

125. Current practice is to limit the total deviation from design alignment to a percentage of the final length of the deep foundation unit; 2% is a common value. However, such practice does not ensure proper structural behaviour of the unit since it does not take into account the length over which this deviation is distributed.
(a) The total deviation from alignment of a deep foundation unit has little influence on its geotechnical capacity unless it reaches values greater than 10% of the length of the unit.
(b) Practically all piles, particularly when driven, are more or less out of design alignment. A straight pile is a theoretical concept seldom achieved in practice.
(c) Only the radius of curvature of a deep foundation unit is important for its structural and geotechnical behaviour. The maximum permissible radius of curvature should be determined by design whenever such radius is required to be measured during inspection. A discussion of permissible bending of piles is given in Reference [42].

Permafrost

126. The lines on Figure K-6 indicate the approximate southern limit of permafrost and the boundary between the discontinuous and continuous permafrost zones in Canada. The distribution of permafrost varies from continuous in the north to discontinuous in the south. In the continuous zone, permafrost occurs everywhere under the ground surface and is generally several decametres thick. Southward, the continuous zone gives way gradually to the discontinuous zone, where permafrost exists in combination with some areas of unfrozen material. The discontinuous zone is one of broad transition between continuous permafrost and ground having no permafrost. In this zone, permafrost may vary from a widespread distribution with isolated patches of unfrozen ground to predominantly thawed material containing islands of ground that remain frozen. In the southern area of this discontinuous zone, permafrost occurs as scattered patches and is only a few metres thick.

127. The lines on this map must be considered as the approximate location of broad transition bands many kilometres wide. Permafrost also exists at high altitudes in the mountains of western Canada a great distance south of the southern limit shown on the map. Information on the occurrence and distribution of permafrost in Canada has been compiled by the Institute for Research in Construction of the National Research Council Canada (now called NRC Construction).[43][44] Special analysis and assessment procedures are necessary for foundation design in permafrost. Such design should only be carried out by professional engineers who are suitably qualified and have the requisite knowledge and experience.

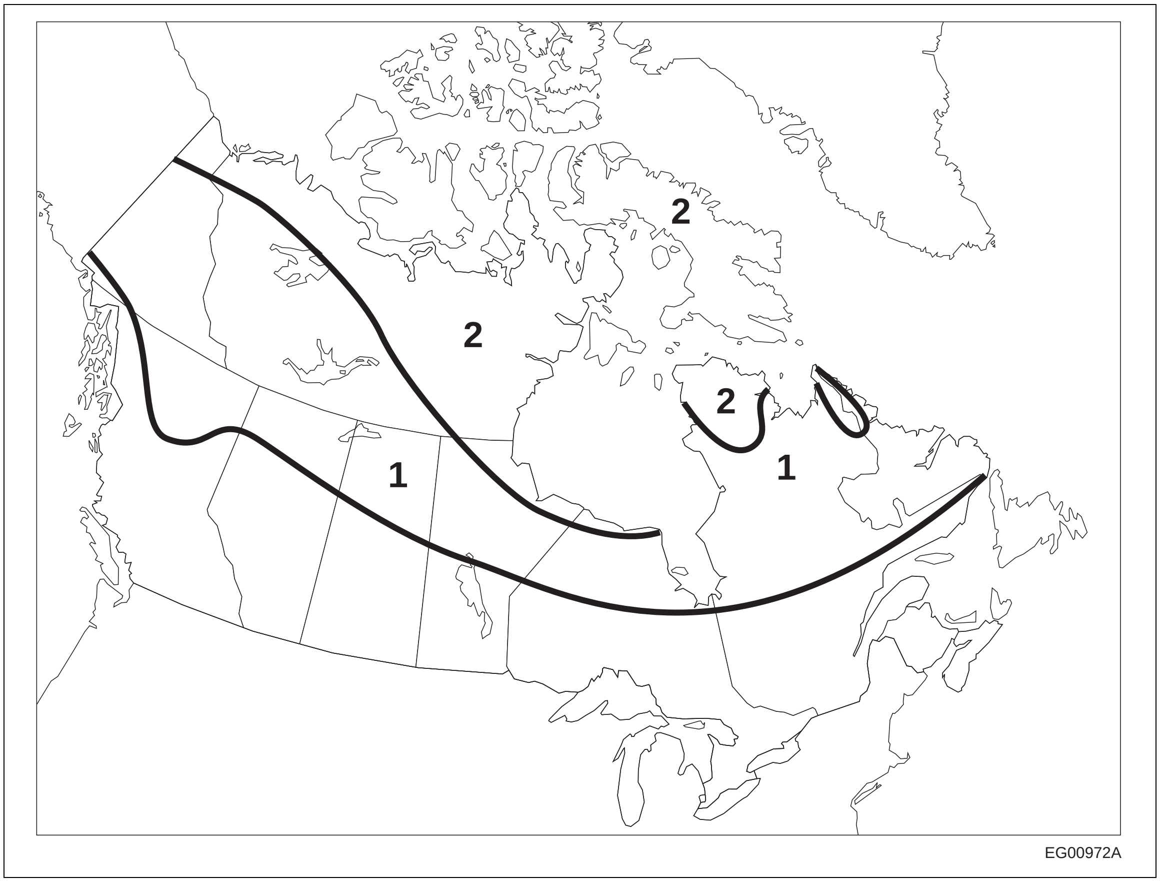

Figure K-6
Discontinuous (1) and continuous (2) permafrost zones in Canada

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Commentary L

Application of NBC Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings

Commentary L

Application of NBC Part 4 of Division B for the Structural Evaluation and Upgrading of Existing Buildings

Notable Change in this Commentary

- Update to the earthquake considerations

Introduction

1. This Commentary provides guidance on the structural evaluation and upgrading of existing buildings to ensure a level of performance that is consistent with the intent of the current National Building Code (NBC) requirements. Buildings that satisfy the guidelines provided in this Commentary are generally considered acceptable. More stringent criteria may be appropriate for buildings used for post-disaster services.

2. This Commentary does not apply to the following:
- new additions, except as provided for in the section titled Earthquakes, and
- the review of newly constructed work required to be in conformance with current codes and standards.

In both of these cases, NBC Part 4 applies without any of the relaxations described in this Commentary. However, it should be noted that new additions can increase the loads on the existing building structure.

3. NBC Part 4 and the structural design standards referenced therein are primarily intended to be used for the design of new buildings (and new additions), not for the evaluation and upgrading of existing buildings. As a consequence, the following difficulties have arisen:
- Many current requirements specify quantities and arrangements of materials (such as reinforcing details in masonry and concrete structures) that can be economically and practically implemented during initial construction but not after a structure is completed. In such cases, alternative solutions are needed.
- Many older buildings have structural systems, components or materials that are not addressed by the structural design standards referenced in NBC Part 4. When properly interconnected, however, older structural systems can be made to work effectively. Because information on their structural properties is lacking, the evaluation and upgrading of such systems is difficult. This difficulty is especially important for heritage buildings.
- Despite their lack of compliance with some aspects of the current NBC and structural design standards, many older buildings have performed satisfactorily over the years without distress or failure. In addition, some structural information, such as dead loads and material properties, can be ascertained for existing buildings by measurement or testing. However, information so ascertained is not taken into account in the structural criteria of NBC Part 4 or the referenced structural design standards.

4. This Commentary aims to help overcome these difficulties by facilitating the application of the requirements of NBC Part 4 to existing buildings through relaxations where appropriate and alternatives where available (usually by reference to other documents). NBC Sentence 4.1.1.5.(2) allows structural alternatives that demonstrate a level of safety and performance in accordance with the requirements of NBC Part 4, but except for load testing, the provisions of this Sentence are directed primarily to new construction. Except as recommended in this Commentary, structural alternatives should comply with the requirements of NBC Sentence 4.1.1.5.(2) (see NBC Note A-4.1.1.5.(2)).

5. Earthquake requirements present the greatest difficulty in the application of NBC Part 4 and the referenced structural design standards to existing buildings. As such, specific guidelines addressing the seismic evaluation and upgrading of existing buildings are presented in the Commentary section titled Earthquakes (Paragraphs 39 to 48).

6. This Commentary does not specify the circumstances that would necessitate the structural evaluation of an existing building, but examples include circumstances where the use of the building changes, where the building experiences damage or deterioration, and where the safety of the building becomes a concern because of known or potential defects.

7. Before any upgrading of an existing building is undertaken, the life safety implications of the conclusions of its structural evaluation should be discussed with the owner of the building and the authority having jurisdiction to decide on a course of action (e.g., to establish a timetable for the work to be done). Each case must be dealt with by taking into account its specific circumstances and the urgency of the upgrading. Examples of actions to be taken include immediate evacuation of the building, a phased repair program, monitoring, further evaluation, and acceptance of the building "as is."

8. Typically, the seismic upgrading of an existing building involves an analysis to determine the weak links in its structural system. The results of this analysis are used to address any weaknesses in the building's Seismic Force Resisting System (SFRS) and to provide a continuous load path from roof to foundation, including both diaphragms and vertical elements. The strength and stiffness of the upgraded SFRS must be compatible with the existing building materials so that weak portions of the vertical-load-carrying system will not be overloaded. Brittle buildings must satisfy lower drift limits than buildings that can deform more flexibly. For example, the drift limit must be more restrictive for a brick building than for a timber-framed building of the same height. Where possible, irregularities that adversely impact the performance of the SFRS should be removed. For example, a seismic-force-resisting brace, a moment-resisting frame or a wall could be added on the open side of a three-sided building. Because the vertical elements of the SFRS must be properly supported, foundation upgrades are often required in seismic upgrading. Another important aspect of seismic upgrading is the fastening of interior and exterior components that could cause injury or block exits if dislodged, such as parapets, brick veneer, unreinforced chimneys, ceilings and unreinforced masonry partition walls.

Basic Considerations

9. The provisions of NBC Part 4 and the referenced structural design standards include general performance requirements and design criteria for buildings, which are based on the following fundamental considerations:
   - life safety,
   - comfort of occupants,
   - function of the building for its intended use,
   - durability, and
   - economics.

10. Life safety, which is the primary consideration, is addressed by design criteria for ultimate limit states (i.e., for strength, stability and integrity). Comfort, function and economics are addressed by design criteria for serviceability limit states, and performance under repeated loads is addressed by criteria for fatigue limit states. Economics are also taken into account by basing the design criteria on appropriate levels of structural reliability to help avoid the unnecessary consumption of materials.

11. The basic considerations of life safety and serviceability apply equally to existing and renovated buildings and to new construction. However, other considerations related to construction costs, user disruption, heritage conservation and resource conservation (e.g., reduction of waste and recycling) may be more critical for existing buildings than for new construction. As a result, structural interventions are usually minimized when existing buildings are upgraded or renovated. Therefore, where it can be shown that the resultant level of life safety (i.e., the probability of death or injury due to structural failure is appropriately low) is generally equivalent to that required by the current NBC and the building is known to be functional, some departure from the current design criteria may be appropriate.

12. Like the design criteria in NBC Part 4 and the referenced structural design standards, the criteria recommended in this Commentary are based primarily on a limit states methodology (see NBC Subsection 4.1.3.). Seismic evaluation requires evaluation of the effects on structural components when the structure is displaced to the drifts expected in an earthquake.

13. This Commentary principally addresses criteria for ultimate limit states, which directly affect life safety. These criteria include the loads, load factors and load combinations specified in NBC Section 4.1., and the resistances and resistance factors specified in the referenced structural design standards. The serviceability and durability problems that can occur as a consequence of a renovation or a change of use or environment are also discussed. Criteria for fatigue limit states, which principally apply to crane-supporting structures, are addressed in CSA S16, "Design of Steel Structures," and the documents referenced therein.

Quality Assurance

14. The design criteria in NBC Part 4 and the referenced structural design standards are based on a level of quality assurance that is consistent with the requirements of NBC Part 1 of Division A and of NBC Part 2 of Division C. The most important of these are the requirements that the designer be a professional engineer or architect skilled in the work concerned (NBC Sentence 2.2.1.2.(1) of Division C) and that the construction of any building or part thereof be reviewed for conformance to the design (NBC Sentence 2.2.7.2.(1) of Division C).

15. These quality assurance requirements also apply to the structural evaluation and upgrading of existing buildings. Indeed, the level of quality assurance may have to be greater for the evaluation and upgrading of existing buildings because the uncertainties concerning their structural properties can be considerably greater than for new construction. More engineering judgment is generally required for the structural evaluation and upgrading of existing buildings than for the design of new buildings. For these reasons, the recommendations in this Commentary are based on the following assumptions:

- that the engineering evaluator has carried out an appropriate structural evaluation of the building and has examined construction details that they consider critical, and
- that the designer will carry out a field review during any upgrading work.

Recommended Codes and Structural Design Standards

16. Recommendations on the codes and structural design standards to be applied in the evaluation and upgrading of existing buildings are summarized in Table L-1. In some situations, it is preferable to use the same standard used to design the building rather than the current standard; an example of such a situation is the evaluation of an old building that was constructed with products that are no longer used, such as undeformed reinforcing bars. Restrictions on the use of earlier codes and structural design standards are given in Table L-1.

Table L-1
Recommended Loads, Load Factors and Structural Design Standards(1)

Application	Current Code/Standards			Current Commentary L	Code/Standards when Built	
	Loads	Load Factors	Structural Design Standards	Load Factors	Loads	Structural Design Standards
Evaluation						
no change in use or occupancy loads	✓	✓	✓	✓	✓ (2)	✓ (2)(3)
change in use or occupancy loads	✓	✓	✓	✓	X	✓ (2)(3)
Design of upgrade	✓	✓ (4)	✓	✓ (4)	X	X

(1) ✓ = acceptable; X = not acceptable.
(2) Acceptable for non-seismic considerations provided the following conditions are met:
- the structure has not experienced significant damage, distress or deterioration;
- the structure was designed and built in accordance with recognized codes and standards; and
- the structure has not been changed in a way that could impair its performance.

(3) Acceptable provided experience has shown that the standard does not present any serious deficiencies.
(4) Current NBC load factors are preferred (see Paragraph 26).

17. Buildings designed and built in accordance with previous codes and structural design standards may be considered acceptable provided the following conditions are met:
- the previous code or standard provides a level of life safety that essentially satisfies the life safety intent of the current edition; and
- neither the building nor its use is altered in a way that affects the building's structural behaviour or increases the loadings on the building.

18. In Table L-2, the benchmark editions of NBC Section 4.1. for use and occupancy loads, for snow, ice and rain loads, and for wind loads are the earliest editions that provide a level of life safety that satisfies the life safety intent of the current requirements for these loads. The requirements for use and occupancy loads, with one or two exceptions, have remained essentially unchanged over the years. In the evaluation of a structural component designed prior to the benchmark year, the current edition of NBC Section 4.1. should be applied using the load factors in NBC Tables 4.1.3.2.-A and -B or those recommended in this Commentary. Alternatively, the evaluation of such a component may be based on satisfactory past performance where the conditions in Paragraph 20 are satisfied.

19. Because earthquake requirements have changed considerably over the years, buildings designed in accordance with earlier codes and structural design standards often do not provide a level of life safety that meets the intent of the current earthquake requirements. The benchmark editions of NBC Section 4.1. for earthquake loads are nevertheless listed in Table L-2 for reference and for assistance in the evaluation of existing buildings as described in the Commentary section titled Earthquakes (Paragraphs 39 to 48).

Table L-2
Benchmark Editions of NBC Section 4.1. and Subsequent Modifications

Loads	Benchmark Editions	Modifications (NBC Edition)
Use and occupancy	1941	guards (1975 and 1995)(1) interior walls acting as guards (1985)
Snow, ice and rain	1960	snow drifts (1965) rain loads – blocked drains (1970) ground snow loads (1990) large flat roofs (1995)
Wind	1960	flexible structures and canopies (1970)
Earthquake	1990(2)(3)	Weak Storey and other structural irregularities (2005) site coefficients (2005) liquefaction (2005)
	2005/2010(2)(3)(4)	new design ground motions (2015) new site coefficients (2015) new M_v and J factors (2015) Gravity-Induced Lateral Demand Irregularity (2015)

(1) The requirements for guard loads in the NBC 1995, which are less stringent than those in the 1975 to 1990 editions of the NBC, should be used in the evaluation of all guards and their supports.

(2) Modern NBC earthquake requirements were introduced in the NBC 1965 and were expanded in the 1977 to 1990 editions of the NBC. Then significant changes were introduced in the NBC 2005 and again in the NBC 2015. See the 2005, 2010 and 2015 editions of Commentary J.

(3) If they satisfy the intent of the modifications listed in this Table, buildings designed in accordance with the earthquake requirements of the NBC 1990 are expected to be in general conformance with the earthquake provisions of this Commentary, but not necessarily with the earthquake requirements of the NBC 2015 and the referenced structural design standards.

(4) Buildings designed in accordance with the earthquake requirements of the NBC 2005 or NBC 2010 are expected to be in substantial compliance with the earthquake provisions of this Commentary. However, such buildings are not expected to be in substantial compliance with the earthquake requirements of the NBC 2015 unless they satisfy the intent of the modifications listed in this Table.

Evaluation Based on Satisfactory Past Performance

20. Buildings and components designed and built according to earlier codes than the benchmark editions, or designed and built in accordance with good construction practice when no codes applied, are considered to have demonstrated a satisfactory capacity to resist loads (other than earthquake loads), provided the following conditions are met:

- careful examination by a professional engineer does not reveal any evidence of significant damage, distress or deterioration;
- the structural system is reviewed, and critical details are examined and checked for load transfer;
- the building has demonstrated satisfactory performance for at least 30 years; and
- there have been no changes within the past 30 years that could significantly increase the loads on the building or affect its durability, and no such changes are contemplated.

21. If these conditions are not satisfied, the evaluation should be based on the recommendations in the Commentary sections titled Load Factors and Load Combinations Recommended for Use in Evaluations (Paragraphs 22 to 27) and Effects Recommended for Use in Evaluations (Paragraphs 28 to 38).

Load Factors and Load Combinations Recommended for Use in Evaluations (NBC Subsection 4.1.3.)

22. Criteria for ultimate limit states should be applied to satisfy the basic objective of life safety. Life safety is described by an acceptable maximum annual probability of death or serious injury resulting from structural failure in a building. This probability is equal to the probability of structural failure (for buildings conforming to NBC Part 4, the probability of structural failure corresponds to a reliability level of approximately 3) times the likelihood of death or serious injury if structural failure occurs. Where the likelihood of death or serious injury is high, there should be no relaxation of the load factors specified in NBC Sentence 4.1.3.2.(2). Where the likelihood is low, as in the case of storage buildings of low human occupancy, the load factors can be reduced; in the NBC, the load factors for such buildings are reduced through the application of an importance factor for the Low Importance Category (see NBC Sentence 4.1.2.1.(3) and NBC Clause 4.1.3.1.(1)(h)).

23. Reduced principal load factors for the structural evaluation of existing buildings, other than post-disaster buildings, are listed according to reliability level and load type in Table L-3. These load factors, which incorporate the principle of an importance factor, are intended to maintain the level of life safety implied by NBC Part 4.[1] The engineering evaluator determines the reliability level using Table L-4 by considering three factors that affect life safety: the behaviour of the structure (system behaviour), the likelihood of people being at risk and the estimated number of people at risk (risk category determined using Tables L-5 and L-6), and the evidence of past safety (past performance). For post-disaster buildings, the loads and load factors of NBC Section 4.1. should be applied.

Table L-3
Principal Load Factors for the Structural Evaluation of Existing Buildings Other than Post-disaster Buildings

Reliability Level(1)	Load Type				
	Dead Load		Live Load(2) or Snow Load	Wind Load	Earthquake Load
	Active	Counteractive(3)			
	Principal Load Factors				
5	1.25	0.90	1.50	1.40	(4)
4	1.20	0.92	1.40	1.30	(4)
3	1.15	0.95	1.30	1.20	(4)
2	1.11	0.97	1.20	1.10	(4)
1 or 0	1.08	1.00	1.00	1.00	(4)

(1) The reliability level is the sum of the indices for system behaviour, risk category and past performance in Table L-4.
(2) A reduction in the live load factor may be justified if the live load in question is controlled (e.g., a liquid in a storage tank); however, the reduced load factor must not be less than the smallest value in the Table.
(3) The counteractive value applies when the dead load acts to resist failure.
(4) See the Commentary section titled Earthquakes (Paragraphs 39 to 48) for guidance on earthquake loads.

Table L-4
Indices for the Determination of Reliability Level for Use with Table L-3

Factor to Consider	Index
System Behaviour	
failure likely to lead to collapse and likely to impact people	2
failure unlikely to lead to collapse or unlikely to impact people	1
failure likely to be local and very unlikely to impact people	0
Risk Category (see Table L-5)	
high	2
medium	1(1)
low	0(1)
Past Performance	
no record of satisfactory past performance	1
record of satisfactory past performance(2) or dead load measured(3)	0

(1) Increase by 1 for assembly areas and for wood structures.
(2) At least 20 years without significant deterioration.
(3) Only applies for the determination of the dead load factor.

Table L-5
Risk Categories for Existing Buildings Other than Post-disaster Buildings for Use with Table L-4

Risk Category	Description
High	Schools and other occupancies where $N \geq 100$(1) Buildings of major heritage importance Industrial and other facilities with hazardous occupancies
Medium	Other occupancies where $5 \leq N < 100$(1)
Low	Other occupancies where the floor area and the adjacent outside area exposed to structural failure are not likely to be occupied by people, and where $N < 5$(1)

(1) The maximum number of people exposed to risk associated with structural failure, N, can be estimated as follows:

N = occupied area exposed to risk, in m^2, × occupant density, in persons per m^2, × duration factor

where
- the occupant density and duration factor can be estimated using the values in Table L-6,
- duration factor = average number of hours of human occupancy per week/100 ≤ 1.0, and
- for outside areas adjacent to the building, the occupant density and duration factor should be estimated in a similar manner.

Table L-6
Parameters for Estimation of N for Use with Table L-5

Primary Use	Occupant Density, Persons per m^2	Average Number of Hours of Human Occupancy per Week
Assembly	1.0	5 – 50
Mercantile, personal services	0.2	50 – 80
Offices, care or detention, manufacturing	0.1	50 – 60
Residential	0.05	100
Storage	0.01 – 0.02	100

24. The engineering evaluator must choose a reduced load factor from Table L-3 for the specific component addressed by the calculation by considering what will happen if it fails and assigning it indices according to Table L-4. Does the structural system have protective features (including non-structural components) that reduce the likelihood of people (both outside and inside the building) being injured or killed in the event of structural failure? Are many people likely to be within the region affected by the failure? For example, the failure of exterior building components (such as masonry parapets) overlooking exits or busy streets presents a greater risk than the failure of components overlooking rarely used areas. Those that fail during an earthquake are generally a greater risk than those that fail in very high winds when fewer people are outside. Finally, if the past performance of an old building is satisfactory, this evidence of its safety can be taken into account for loads other than earthquake loads.

25. Table L-5 provides guidance on determining the risk category used in Table L-4, including a procedure for estimating the number of people exposed to risk associated with the structural failure. In applying this procedure, the engineer should estimate the area of the building that is likely to be affected by the failure mode of the component being evaluated. For example, a punching shear failure is likely to cause a total collapse of a flat slab building, whereas a floor joist failure is likely to affect only a small area of the building.

26. While the reduced load factors in Table L-3 are intended to maintain a low risk to life safety, they correspond to an increased risk of building damage due to structural failure. They should be considered as a minimum performance level, which, if not met, indicates the need for upgrading. They may not be appropriate for use in the design of the upgrade. Where the difference in upgrading cost due to increasing the minimum load factor is small and the potential loss due to failure is large, higher load factors, such as those specified in NBC Sentence 4.1.3.2.(2), are recommended for the design of the upgrade. The level of upgrading should be determined in consultation with the owner.

27. The load combinations specified in NBC Tables 4.1.3.2.-A and -B should be used in the evaluation, with only the principal load factors reduced in accordance with Table L-3.

Effects Recommended for Use in Evaluations

28. Because the effects specified in NBC Part 4 primarily address ultimate limit states and life safety, relaxations of these effects are generally not recommended. In some cases, however, it is possible to determine loads more accurately for evaluation than for design. Earthquake loads are discussed in the Commentary section titled Earthquakes (Paragraphs 39 to 48).

Effects Due to Movements, T (NBC Sentences 4.1.2.1.(1) and 4.1.3.2.(4))

29. Effects due to movements caused by temperature changes, moisture changes and sustained stress (e.g., shrinkage, creep and differential settlement) can usually be ignored in the structural evaluation of an existing building, provided an inspection of components and connections does not uncover any damage affecting the safety of the building. The past performance of the existing building will show whether such movements caused local damage or displacements that affected its strength or integrity. Ten years of past performance is usually sufficient, but for the differential settlement of footings on materials such as clay, approximately 30 years is necessary.

30. In the upgrading of an existing building, consideration should be given to differential movements between the new and old materials.

Dead Loads, D (NBC Subsection 4.1.4.)

31. Where dead loads are determined from field measurements, the uncertainty of the dead loads used in evaluation is lower than that of the dead loads used in design. Tables L-3 and L-4 take this into account by means of a reduction in the dead load factor. Similarly, Note (2) to Table L-3 allows a reduction in the live load factor where the live load is controlled.

32. Due to the difficulty in controlling future installations of partitions in office buildings, it is recommended that a partition weight of 1 kPa, as specified in NBC Sentence 4.1.4.1.(3), be allowed for in such occupancies.

Live Loads Due to Use and Occupancy, L (NBC Subsection 4.1.5.)

33. Loads due to people, such as those for assembly, access and exit areas, have a direct effect on life safety. Note (1) to Table L-4 therefore allows less of a reduction in the load factors to be applied to loads in low- and medium-risk assembly areas in Table L-3.

34. In an existing building, it may be possible to restrict some floor loads to a lesser value than that specified in NBC Subsection 4.1.5. If the analysis of the projected use of the floor area clearly indicates that the NBC load, including dynamic effects, will not be approached, then a reduction in load may be warranted, provided that any future change of use is restricted. For example, NBC Article 4.1.5.6. allows a reduction of the minimum specified live load of 4.8 kPa for dining areas to 2.4 kPa for areas in buildings that are being converted to dining areas, provided the floor area is 100 m^2 or less and the dining area will not be used for other assembly uses, such as dancing. However, because future use is generally difficult to control, this provision should be used with caution and only with the approval of the authority having jurisdiction.

35. The provision of NBC Sentence 4.1.3.6.(2) requiring the dynamic analysis of floors supporting an assembly occupancy used for rhythmic activities need not be applied to the floor of an existing building, provided vibration of the floor has not been distinctly noticeable in the past and no change of use of the floor is contemplated.

36. For all other use and occupancy loads, it is recommended that NBC Part 4 be followed.

Loads Due to Snow and Rain, S (NBC Subsection 4.1.6.)

37. It is generally difficult to justify a reduction in snow and rain loads from those specified in NBC Subsection 4.1.6. and recommended in Commentary G. However, many years of satisfactory roof performance despite apparent structural deficiencies in relation to the current NBC requirements may indicate a need to better assess actual snow loads on the building. Special studies including a comparison of ground snow accumulations measured at the building site with those measured at the Environment and Climate Change Canada weather station, as well as special model or analytical studies of snow accumulation on the building in its location, can be used to more closely estimate the site-specific snow load. The assumptions of such studies may not apply, however, if there is a change in roof geometry or in wind exposure (e.g., due to new buildings). A change in the snow loads on an existing building can also occur due to changes in insulation or indoor heating, or due to snow sliding off a sloping roof as a result of a change in roofing material. See Commentary G for further guidance.

Wind Loads, W (NBC Subsection 4.1.7.)

38. It is equally difficult to justify a reduction in wind load from that specified in NBC Subsection 4.1.7. and recommended in Commentary I. However, many years of satisfactory performance despite apparent structural deficiencies in relation to the current NBC requirements may indicate the need to better assess actual wind loads on the building. Special studies including a comparison of wind speeds measured at the building site with those measured at the Environment and Climate Change Canada weather station, as well as model or analytical studies of wind loads on the building in its location, can be used to more closely estimate the site-specific wind load. The assumptions of such studies may not apply, however, if there is a change in building shape or local topography. See Commentary I for further guidance.

Earthquakes (NBC Subsection 4.1.8.)

39. The current earthquake requirements of NBC Part 4 and the referenced structural design standards can present major difficulties for the rehabilitation of existing buildings, particularly heritage and other buildings constructed of unreinforced masonry. It is relatively easy to satisfy the current earthquake requirements in the construction of new buildings; however, while the goal of rehabilitation is to fully upgrade existing buildings to meet the current earthquake requirements, it is usually very disruptive, costly and difficult to do so. In many cases, because changes have been made to the NBC and the referenced structural design standards since the building was constructed, it is almost impossible to satisfy all the requirements in every detail. Often, the rehabilitation does not proceed and the building remains unchanged to avoid the attendant costs and disruption (e.g., the potential loss of use during the renovation). In effect, the requirement for a full seismic upgrade to current Code requirements becomes a disincentive to any improvement. For this reason, many documents recommend less rigorous earthquake provisions for upgrading existing buildings, which facilitate building improvement without acting as a deterrent: e.g., previous editions of this Commentary, ASCE/SEI 41, "Seismic Evaluation and Retrofit of Existing Buildings," and many FEMA/NERHP documents.

40. Some buildings under renovation, particularly heritage buildings built of non-ductile materials, have a very low resistance to seismic forces or to the drift imposed by seismic forces and are a significant hazard. Renovations that add mass to an existing building, that increase the irregularity of the building, or that increase the height of the building will increase the risk of collapse. Renovations that extend the life of an existing building increase the duration for which the occupants are exposed to risk. All these cases are candidates for seismic upgrading.

41. At present there is no requirement in Canada to seismically upgrade a building that is not being renovated or expanded. However, an owner may consider a voluntary seismic upgrade to mitigate the risk to occupants and to improve the likelihood of continuity of service during and after minor seismic events. Many jurisdictions that have recently experienced the effects of earthquake damage, such as California and New Zealand, have mandatory seismic upgrading requirements for buildings with earthquake-resistance issues such as unreinforced masonry, weak stories, non-ductile concrete, and parapets.

42. Although the earthquake provisions in this Commentary do not always provide the level of performance and safety expected for a new building, they should encourage the seismic upgrading of many more buildings to achieve improved safety and performance. The flow chart in Figure L-1 provides suggestions to be applied in the seismic assessment and upgrading of existing buildings that are being renovated, expanded or voluntarily upgraded. Authorities having jurisdiction can use these suggestions to develop their own requirements. It is expected that, during any renovation or expansion of a building with a deficient structural system, at least some of the project budget should be spent on seismic upgrading. The level of seismic upgrading required will depend on the extent of the renovation or expansion and on the current lateral force resistance of the building. Note that a voluntary life safety system upgrade (i.e., a non-mandatory upgrade of the life safety systems in the existing building, which includes upgrades to sprinklers, fire and smoke alarms, and means of access) does not trigger a seismic upgrade.

43. In this Commentary, the seismic assessment and upgrading of existing buildings is performed according to appropriate assessment/upgrading levels, each of which corresponds to design ground motions with a specified return period, as defined in Note (1) to Figure L-1. The intent of the seismic assessment and upgrading is to ensure that the SFRS of the existing building is compatible with a desired level of risk. To fully comply with an assessment/upgrading level, the building must be able to withstand the seismic load for that level and the drift imposed by this seismic load. The degree of compliance is determined by considering both the strength (i.e., lateral-load-resisting capacity) and the drift capacity of the building. The strength of the building is compared to the earthquake load for the applicable level to determine the percentage force compliance. Where failure of the vertical-load-carrying system of the building occurs at a drift smaller than that defined in the NBC, the force causing the drift is compared to the earthquake load for the applicable level to determine the percentage drift compliance. The degree of compliance is taken as the lower of the percentage drift compliance and the percentage force compliance.

44. In evaluating force compliance, the earthquake load for the applicable level and the strength of the existing building are determined using appropriate R_dR_o values and material factors from the current NBC and structural design standards. If the building has an SFRS that is substantially equivalent to an SFRS having a defined R_dR_o value, then this value is used. For buildings having an SFRS that is not defined in the current NBC or structural design standards, an R_dR_o value given in a seismic rehabilitation standard such as ASCE/SEI 41 may be used. A value of $R_dR_o = 1.0$ is used for an SFRS with little ductility (e.g., unreinforced masonry). ASCE/SEI 41 may be used as an alternative to the guidance in this Commentary, provided the ground motions used are as per the NBC 2015 for the return periods suggested in this Commentary and the intent of the NBC 2015 is satisfied. In evaluating drift compliance, a value of $R_dR_o = 1.0$ is used.

45. Provisions that cause difficulties for seismic upgrading include those specifying restrictions on structural systems (NBC Sentence 4.1.8.9.(1) and NBC Article 4.1.8.10.), restrictions related to lateral deflections and pounding (NBC Articles 4.1.8.13. and 4.1.8.14.), and restrictions on detailing for earthquake effects (referenced structural design standards). To help overcome these difficulties, it is recommended that a standard such as ASCE/SEI 41 be used as a guide. However, judgement will need to be exercised in order to take into account the seismic provisions of the NBC and this Commentary and the detailing requirements of the structural design standards.

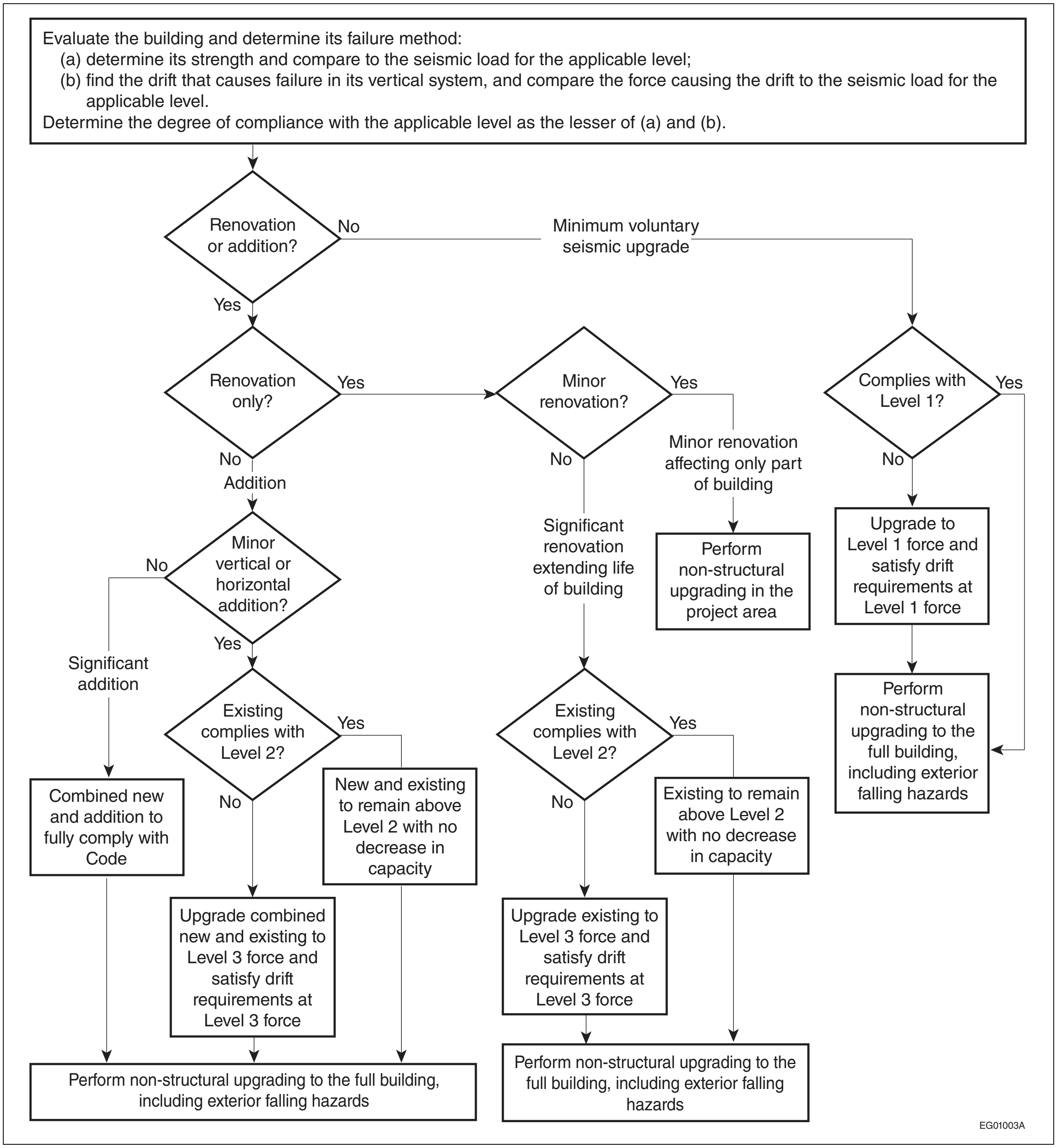

Figure L-1
Flow chart for the seismic assessment and upgrading of existing buildings

Notes to Figure L-1:

(1) The following assessment/upgrading levels are used in the seismic assessment and upgrading of existing buildings:

Level 1: This assessment/upgrading level is for minimum voluntary seismic upgrades. An evaluation of the SFRS must be performed and deficiencies such as weak storeys, discontinuities in the SFRS, inadequate capacity, excessive irregularity including torsional eccentricity, and incomplete lateral load paths must be identified. The upgrade must address these deficiencies as a priority and must also address the restraint of falling hazards, such as parapets. The use of spectral response acceleration values corresponding to 0.5 times those with a probability of exceedance of 5% in 50 years (1/1 000 per year) is suggested.

Level 2: For this assessment/upgrading level, the use of spectral response acceleration values with a probability of exceedance of 10% in 50 years (1/475 per year) is suggested.

Level 3: For this assessment/upgrading level, the use of spectral response acceleration values with a probability of exceedance of 5% in 50 years (1/1 000 per year) is suggested.

The 5%-in-50-year and 10%-in-50-year spectral response acceleration values can be obtained by using the "Hazard Calculator" on the Earthquakes Canada Web site of the Geological Survey of Canada (www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2015-en.php).

The levels of force are intended to be used with the rest of the earthquake design provisions in NBC Subsection 4.1.8.

(2) Description of terms used in seismic assessment and upgrading:

Addition: An addition is structurally connected to the existing building. Structures that are structurally separated from the existing building are considered separate buildings.

Horizontal Addition: A horizontal addition is an addition that increases the area of the building without increasing its height and that may or may not increase the footprint of the building.

Major renovation: A major renovation is an extensive renovation to the architectural, structural, mechanical and electrical components in a major portion of the building that extends the useful life of the building. The renovation may or may not involve removal of the wall and ceiling finishes in the project area. A change of use is also considered a major renovation.

Minor addition: A minor addition is an addition having a total weight that is less than 10% of the weight of the existing building.

Minor renovation: A minor renovation is a limited renovation to the architectural, mechanical and electrical components in a portion of the building. The renovation may or may not involve some structural work, but does not increase the occupied area of the building. A minor renovation is limited to one floor in a building with three or more storeys and to a part of one floor in a one- or two-storey building; a renovation affecting a larger part of the building is considered a major renovation. Minor renovations must not reduce the capacity of the SFRS.

Minor renovation involving structural components: A minor renovation involving structural components is a minor renovation that involves a change to the structure of the existing building (e.g., a renovation that creates an additional opening in a shear wall). The renovation may increase the vertical or lateral capacity of the existing building, but must not reduce its lateral capacity.

Non-structural upgrading in the project area: Non-structural upgrading in the project area addresses the required restraint of ceilings, mechanical and electrical equipment and components, and partitions, including unreinforced masonry partitions. The upgrading extends over the project area; for example, if one floor of a multi-storey building is being renovated, then that floor requires non-structural upgrading. For major renovations, the project area should include all exterior falling hazards, such as parapets, cornices, glazing, architectural exterior panels, canopies, statues, terracotta, and other ornamentation. All non-structural upgrading must satisfy all the requirements of NBC Article 4.1.8.18.

Non-structural upgrading to the full building, including exterior falling hazards: Non-structural upgrading to the full building addresses the required restraint of ceilings, mechanical and electrical equipment and components, and partitions, including unreinforced masonry partitions. The upgrading extends over the entire building and addresses all exterior falling hazards, such as parapets, cornices, glazing, architectural exterior panels, canopies, statues, terracotta, and other ornamentation, regardless of the extent of the renovation. All non-structural upgrading must satisfy all the requirements of NBC Article 4.1.8.18.

Vertical Addition: A vertical addition is an addition that increases both the area and the height of the building with or without increasing its footprint. Vertical additions are usually built at least in part on the existing building. However, a structurally connected addition that is taller than the original building without being on the footprint of the original building is also considered a vertical addition.

Voluntary seismic upgrade: A voluntary seismic upgrade is a non-mandatory upgrade of the SFRS. Upgrading to Level 1, the minimum assessment/upgrading level, is recommended. Non-structural upgrading is also recommended.

46. In doing a seismic review of a building or in determining its compliance with the current NBC, it is important to review both the capacity of the SFRS to carry seismic loads and the ability of the vertical-load-carrying system to accommodate the deformations that the seismic loading will impose. For brittle structures, such as buildings with multi-wythe brick walls or non-ductile concrete frames, drift requirements often govern. Information on the drift capacities of various types of construction can be found in ASCE/SEI 41. Additional information on concrete elements can be found in CSA A23.3, "Design of Concrete Structures."

47. For many buildings in regions of low to moderate seismicity (where $S_a(0.2) \leq 0.75$), life safety can be greatly improved at relatively low cost by providing lateral support to masonry and other heavy non-structural components.

48. In most seismic regions of Canada, particularly those in the east, non-structural building components have posed a greater risk in recent earthquakes than the building structures themselves. Also, seismic upgrading can often be carried out much more easily for non-structural components than for the building structure—as part of maintenance. It is recommended that CAN/CSA-S832, "Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings," be followed for the seismic upgrading of non-structural components.

Serviceability

49. The serviceability requirements of NBC Part 4 (NBC Articles 4.1.3.4. to 4.1.3.6. and much of NBC Section 4.2.) and the referenced structural design standards address human comfort and the function of the building structure for its intended use (e.g., operation of equipment, drainage, and protective function of the building envelope).

50. These serviceability criteria are intended for the design of new buildings. For existing buildings, in many cases, the demonstration of satisfactory past performance eliminates the need to apply the criteria in structural evaluation. Unacceptable deformation, settlement, vibration, or local damage will usually be evident to the occupants of an existing building within a period of 10 to 30 years after its construction. However, a serviceability evaluation may be required where there is a change of use or an alteration of building components that affects the properties of the structure, for example.

51. An example of a change of use is the introduction of an activity such as aerobics or jogging into an existing building. In this case, the existing floor structure should be evaluated for the new use by means of either a performance test or calculation procedures (see Commentary D for further guidance). An evaluation is also recommended for intended new uses such as the installation of reciprocating machinery and the operation of equipment that is sensitive to vibration, floor smoothness or slope.

52. An example of an alteration of building components that affects the properties of the structure (and therefore its response to loading) is the removal of partitions, which reduces the damping and stiffness of the floor system and increases its sensitivity to vibration induced by footfalls. In this case, it is recommended that the floor construction be reviewed for the intended use before removing the partitions. Similar alterations that can affect structural serviceability include changes to cladding and partitions in tall buildings, which affect wind sway motions, and the addition of heavy components, which results in increased deflection.

53. With respect to earthquake criteria, the interstorey deflection limits of NBC Sentence 4.1.8.13.(3) are intended to control damage to non-structural components, but whether they accomplish this goal will usually not have been tested by experience. For guidance, see Reference [2].

Durability

54. Durability is a major factor affecting serviceability and safety, which, although not addressed in the design criteria of NBC Section 4.1., is addressed in NBC Section 4.2. and in the structural design standards referenced in NBC Sections 4.3. and 4.4. (often by reference to other standards, such as CSA A23.1, "Concrete Materials and Methods of Concrete Construction"). CSA S413, "Parking Structures," which is referenced in NBC Sentence 4.4.2.1.(1), is essentially concerned with durability, as are CSA S448.1, "Repair of Reinforced Concrete in Buildings and Parking Structures," and CSA S478, "Guideline on Durability in Buildings."

55. Corrosion failures of unbonded post-tensioned beams and slabs, reinforced concrete parking structures, supports and connections for precast and other wall panels, masonry wall ties, and deep foundations can result in unsafe structures without visible deterioration. ASCE/SEI 11, "Guideline for Structural Condition Assessment of Existing Buildings," and Reference [3] provide guidance on the assessment of such conditions.

56. A change of use (e.g., with a change in internal environmental conditions) or an alteration of building components (e.g., insulation) may result in future deterioration where none occurred in the past, particularly to exterior wall components. Such potential deterioration should be considered in the evaluation.

Structural Integrity

57. In the structural evaluation of an existing building, the engineering evaluator should consider the ability of the structure to absorb local failures without widespread collapse . This important property can be assessed by considering the likelihood of specific failures due to overloading, accidental damage, defects and deterioration, and, if the likelihood exists, by considering the ability of the building (both structural and non-structural components) to provide alternative paths of support. The latter consideration, however, is not easily quantifiable and therefore involves considerable engineering judgment. Table L-4 takes alternative paths into account by means of a reduction in the system behaviour index where such paths exist, which results in a reduction in the load factors in Table L-3. See Commentary B for further guidance.

Foundations

58. The adequacy of spread footings can generally be demonstrated by satisfactory past performance. Consideration should, however, be given to spread footings that will be subjected to a significant increase in loading. Consideration should also be given to deep foundations in situations where they may have been weakened by deterioration.

59. Guidance concerning the effects of earthquakes on foundations is given in CSA A23.3.

Referenced Structural Design Standards

60. In the application of the structural design standards referenced in NBC Section 4.3. to existing buildings, the engineering evaluator is advised to follow the ultimate limit state requirements for resistance (including resistance factors) contained in each standard. Information contained in CSA S6, "Canadian Highway Bridge Design Code," may be helpful.

61. Alternatively, the building may be considered adequate on the basis of satisfactory past performance, provided the conditions described in Paragraph 20 are met.

62. The Commentary section titled Load Testing (Paragraphs 63 to 69) provides guidance on determining resistance by means of load tests as an alternative to structural analysis.

Load Testing

63. Load testing can be used for structural evaluation where safety is in doubt (e.g., due to a lack of drawings or design information, deterioration, fire, or possible inherent deficiencies). In some cases, load testing can be used to monitor the effects of deterioration (see Reference [4] for guidance). Load testing is generally used as a last resort in the structural evaluation process, because it is usually disruptive and costly.

64. The load testing of existing building structures mainly consists of proof tests to establish safety. Occasionally, it may be useful to carry out destructive ultimate load tests of isolated structural components to determine their capacity and mode of failure. Load tests can also be used to determine component forces in a structure where it is difficult to apply a conventional structural analysis.

65. In some situations, a load test may not provide sufficient evidence concerning the future safety of the structure. An example of such a situation is a post-tensioned structure with very little normal reinforcement where there is hidden corrosion of the prestressing tendons. Although such a structure may pass a load test, further deterioration may result in a sudden brittle failure.

66. It is important that the structure be exposed and accessible for visual inspection before, during and after a load test.

67. For proof tests, the loads should be applied to the structure in a pattern representative of the expected loading and in a manner producing the maximum effects for the critical modes of potential failure as ascertained by the evaluator. The proof test loads should be representative of the effects of factored loads specified in NBC Section 4.1., or some multiple thereof, depending on the type of failure (e.g., gradual versus sudden) and on whether the whole structure or only a representative portion is tested. For concrete structures and composite concrete and steel structures, the requirements of CSA A23.3 should be observed. In the case of non-composite steel frame structures, an evaluation can normally be done by measurement and calculation. For structures made of other materials, a test load (including the weight of the structure tested) representing 1.3 times the total dead load of the renovated building plus 1.6 times the live load should be applied for a minimum of 24 hours. The test should include the measurement of deflections and of recovery after the load is removed.

68. In general, the structure is considered to pass the load test if there is no evidence of impending failure during the test. However, evidence of excessive cracking or deflection (short- or long-term) under specified loads may indicate serviceability problems, which should be evaluated by considering the past performance of the structure and any contemplated change of use.

69. For more guidance on load testing, see Reference [4].

Commentary L

Further Guidance on Methods of Structural Evaluation

70. Further guidance on methods of structural evaluation is contained in References [5] and [6].

References

[1] D.E. Allen, Criteria for Structural Evaluation and Upgrading of Existing Buildings. Can. J. Civ. Eng., Vol. 17, No. 6, December 1991.

[2] D.E. Allen, J.H. Rainer and A.M. Jablonski, Guidelines for the Seismic Evaluation of Existing Buildings. Institute for Research in Construction, National Research Council Canada, Ottawa, 1992. NRCC 36941.

[3] N.R. Webster, Evaluation of Unbonded Post-Tensioned Structures. Proc. of Second Canadian Symposium on Cement and Concrete. Sidney Mindess, ed. University of British Columbia, Vancouver, British Columbia, 1991, pp. 230-241.

[4] Load Testing of Structures and Structural Components. Institution of Structural Engineers, London, England, 1989.

[5] Evaluation, Maintenance and Upgrading of Wood Structures: A Guide and Commentary. Am. Soc. Civ. Eng., New York, New York, 1982.

[6] Structural Renovation of Traditional Buildings. Construction Industry Research and Information Association, CIRIA Report III, London, England, 1986.