cursus

ELEMENTS OF EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS

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Preface

Major earthquakes have occurred throughout history and continue to occur today all over the world. Since the occurrence and severity of earthquakes cannot yet be predicted, earthquake engineering needs to be more widely applied to reduce the seismic risk facing many densely populated regions worldwide, including Canada and the United States. Within North America, the earthquake hazard varies from region to region and the current literature on earthquake engineering can be difficult to grasp for structural engineers who have not had previous training in seismic design. Furthermore, no single resource currently addresses seismic design practices in both Canada and the United States. This book was written to fill the gap.

In coordinating the writing of this book, it has been a privilege to benefit from the talent, expertise and diverse professional experiences of this book's six co-authors: Dr. Robert Tremblay, Dr. Constantin Christopoulos, Dr. Jeffrey Erochko, Dr. Bryan Folz, Dr. Didier Pettinga and Dr. Pedram Mortazavi. The book presents the key elements of earthquake engineering and structural dynamics at an introductory level, while emphasizing the seismic design provisions currently in use in Canada and the United States. This fourth edition of *Elements of Earthquake Engineering and Structural Dynamics* gives readers the basic knowledge they need to apply the seismic provisions contained in building codes in Canada and the United States. Earthquake engineering is a multidisciplinary science, and the scope of this book is not limited to structural analysis and design; it also discusses the basics of other relevant topics such as seismology, seismic risk analysis and geotechnical engineering to help structural engineers work efficiently with other specialists on earthquake-resistant construction projects. The authors hope that practising engineers, senior undergraduate and graduate structural engineering students and university educators will all find this book a useful resource.

In Chapter 1, basic concepts of earthquake engineering are presented along with a brief overview of the history of earthquake engineering and its major milestones in the United States and Canada. Chapter 2 discusses the seismological aspects of earthquakes that are the most relevant to structural engineering. Chapter 3 describes deterministic and probabilistic seismic hazard analyses with an emphasis on how seismic hazard maps are constructed based on historical data. The major assumptions behind the development of the seismic hazard maps assembled for the entire Canadian and American territories are also discussed. Chapter 4 presents the fundamental notions of linear and nonlinear dynamic analyses of structures with a particular emphasis on the response of structures subjected to earthquake ground motion input. These same concepts of dynamic analysis are used in Chapter 5 to evaluate the influence that a soil deposit may have on the movement at the base of a structure; this chapter also addresses the problem of the liquefaction of saturated granular soils and the soil-structure interaction phenomenon.

The next four chapters, 6 through 9, are aimed at practising structural engineers who must account for the effects of earthquakes in the design of their structures. Chapter 6 provides a detailed presentation of the seismic provisions included in the 2020 edition of the National Building Code of Canada and the 2022 edition of the American Society of Civil Engineers ASCE 7 standard in the United States. The chapter focuses on the concept of ductile structural response that is at the core of the seismic design philosophy in modern seismic design codes. Chapter 7 describes the specific seismic detailing requirements that must be incorporated into the design of steel structures in Canada and the United States. Similarly, the seismic design provisions required for the design of reinforced concrete structures in Canada and the United States are described in Chapter 8. Finally, Chapter 9 provides an overview of the general seismic design and detailing requirements for light-frame wood buildings, which are the most common type of buildings in Canada and the United States, and for mass timber construction, which is rapidly emerging.

The last two chapters of the book address special topics of importance in earthquake engineering: in Chapter 10, the current regulations and guidelines for the seismic design and specifications of nonstructural building components in Canada and the United States, and in Chapter 11, a brief overview of the common supplemental damping and seismic isolation systems that have been implemented to raise the seismic performance of structures.

Each chapter ends with a set of problems to help readers learn the practical aspects of the subject. This book can be used as a reference for a senior undergraduate course or a junior graduate earthquake engineering course or as a self-teaching manual for practising engineers.

I would like to first thank the six co-authors of the book for having so generously shared their expertise and experience during this book project—specifically, the expertise of Dr. Pedram Mortazavi and Dr. Robert Tremblay in steel design, Dr. Constantin Christopoulos in dynamics and seismic analysis methodologies, Dr. Jeffrey Erochko and Dr. Bryan Folz in wood design and Dr. Didier Pettinga in reinforced concrete design. They had a profound impact on the scope, technical level and completeness of the material presented in this book.

I would also like to thank Presses internationales Polytechnique for publishing this book, particularly Luce Venne-Forcione, Martine Aubry, Virginie Vendange and Constance Forest, as well as Isabelle Leclerc and Flavio Mini.

In addition, I would like to thank many colleagues from the University of British Columbia, Polytechnique Montréal, the University of California at San Diego, the State University of New York at Buffalo and the Scuola Universitaria Superiore IUSS di Pavia (University School for Advanced Studies IUSS Pavia) in Italy for their technical collaborations, discussions and exchanges over the years, which have all positively influenced the final outcome of this book.

Similarly, I would like to thank the 72 graduate students that I have had the immense pleasure of supervising over the years for giving me a continuous source of motivation and the energy to coordinate the writing of this book.

Finally, I would like to express my deepest gratitude to my doctoral supervisor, the late Professor Emeritus Shel Cherry at the University of British Columbia, for introducing me to earthquake engineering research. His professionalism, enthusiasm and broad-mindedness continue to be a source of inspiration well beyond the bounds of my technical activities.

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Biographical Notes

André Filiatrault, P.Eng., Ph.D., is a professor emeritus in the Department of Civil, Structural and Environmental Engineering at the State University of New York at Buffalo in Buffalo, NY. He received his master's (1985) and Ph.D. (1988) degrees in civil engineering from the University of British Columbia after obtaining his bachelor's degree in civil engineering from Université de Sherbrooke in 1983. After a two-year stint as an assistant professor at the University of British Columbia, he joined the Department of Civil, Geological and Mining Engineering at Polytechnique Montréal in Montréal, Canada, where he became a full professor in 1997. Professor Filiatrault joined the faculty at the University of California, San Diego in 1998, where he was a professor of structural engineering until 2003. In 2003, he became a professor in the Department of Civil, Structural and Environmental Engineering at the University at Buffalo (UB), State University of New York. From 2003 to 2007, he served as the deputy director of the Multidisciplinary Center for Earthquake Engineering Research (MCEER), and as director from 2008 to 2011. In 2014, he joined the Scuola Universitaria Superiore IUSS di Pavia (University School for Advanced Studies IUSS Pavia) in Italy, where he remained a full professor until 2023. His 35-year research career focused on the seismic testing, analysis and design of civil engineering structures and nonstructural components. The professional achievements resulting from his research and teaching activities include five textbooks, more than 400 peer-reviewed scientific publications, the 1990 Sir Casimir Stanislaus Gzowski Medal from the Canadian Society for Civil Engineering, the 2002 Moisseiff Award from the American Society of Civil Engineers (ASCE), the 2008 Outstanding Researcher/Scholar Award from the Research Foundation of the State University of New York and the Best Paper in Analysis & Computation Award for 2021 from the ASCE Journal of Structural Engineering.

Constantin Christopoulos, Ph.D., P.Eng., is a professor of civil engineering at the University of Toronto (U of T) in Toronto, Canada. He received his Ph.D. from the University of California at San Diego in 2002 after completing his bachelor's and master's degrees at Polytechnique Montréal. He is the director of the Structural Testing Facilities at U of T and held the Canada Research Chair in Seismic Resilience of Infrastructure from 2012 to 2022. His research is primarily focused on the development of advanced high-performance structural systems that enhance the dynamic response and seismic resilience of built infrastructure. He is the author of more than 200 technical papers and is named as an inventor on multiple issued international patents. He has received multiple awards for his innovative developments in the field of earthquake engineering. He is an associate member of the CSA S16 Technical Committee on Steel Structures and is also actively involved in the development and transfer into the market of new technologies developed through research conducted at U of T. He is a co-founder of two U of T spinoff companies: Cast Connex, which specializes in cast steel connections and casting design, and Kinetica, which specializes in the design and implementation of solid-state dampers in civil structures.

Didier Pettinga, Ph.D., P.Eng. C.P.Eng., is a technical director with Holmes NZ in Christchurch, New Zealand, and professor of practice at the University of Canterbury, where he focuses on structural design practice and assessment and retrofit methods. He graduated in 2002 with a Bachelor of civil engineering from the University of Canterbury, New Zealand. He completed his M.Sc. and Ph.D. in earthquake engineering at the European School for Advanced Studies in Reduction of Seismic Risk (ROSE School) from 2003 to 2006. From 2007 to 2011 he was based in Vancouver, Canada, as an earthquake engineering specialist with Glotman Simpson Consulting Engineers, providing structural design practice experience in both Canada and the West Coast of the United States. Since 2012, he has been based in Christchurch and has worked on a number low-damage seismic design projects using seismic base isolation and supplemental damping.

Jeffrey Erochko, Ph.D., P.Eng., is an associate professor of civil engineering at Carleton University in Ottawa, Canada. He completed a bachelor's degree in engineering science in 2006 and a Ph.D. in civil engineering in 2013, both at the University of Toronto. His research is focused on earthquake-resistant design for timber, steel, concrete and unreinforced masonry buildings, the development of high-performance seismic-resistant systems and the application of hybrid simulation for earthquake and fire testing. Dr. Erochko is the co-principal investigator for the $7.5 million Earthquake and Multi-Hazard Test Facility for Built Infrastructure Protection and Resilience at Carleton (a joint facility with the University of Ottawa) and is currently the associate director of the Ottawa-Carleton Institute for Civil Engineering. He is the author of a textbook on structural analysis, and he has taught timber design to undergraduate engineering students for over ten years. Over that time, he has won numerous awards for teaching, including the Carleton University Teaching Achievement Award, and in 2022 he was appointed as a Provost's Teaching Fellow.

Pedram Mortazavi, Ph.D., P.Eng., is an assistant professor in the Department of Civil, Environmental, and Geo- Engineering at the University of Minnesota. He is also currently serving as the co-director of the Galambos and Multi-Axial Subassemblage Testing (MAST) Laboratories at the University of Minnesota. He received his Ph.D. in civil engineering from the University of Toronto in 2023, and his master's degree from Carleton University in 2014.

Dr. Mortazavi has more than six years of industry experience having worked as a consultant at several consulting and R&D firms. His current research is focused on leveraging his expertise in steel structures, large-scale testing and advanced simulation methods such as hybrid simulation toward developing high-performance structural components and systems that enhance the performance and resilience of structures after extreme events. Dr. Mortazavi is a member of the CSA S16 Technical Committee on Steel Structures, a regular participant at the American Institute of Steel Construction (AISC) Task Committee 9 on Seismic Design and is a member of several Canadian Institute of Steel Construction (CISC) task groups for seismic design of structures. Dr. Mortazavi is currently the vice-chair of Task Group 06: Extreme Loads of the Structural Stability Research Council (SSRC) and is a member of the editorial board of the Resilient Cities and Structures Journal. He has received many recognitions including the G.J. Jackson Fellowship by CISC, the Donald Jamieson Fellowship by CSCE and the TA-TATP Teaching Excellence Award at the University of Toronto, among others.

Bryan Folz, Ph.D., P.Eng., was a faculty member in the Department of Civil Engineering at the British Columbia Institute of Technology (BCIT) in Vancouver, Canada, from 1993 to 2017. He received a Ph.D. in civil engineering from the University of British Columbia in 1997. His research has focused on timber engineering, reliability analysis, structural modelling and seismic design. Dr. Folz's professional achievements in research and teaching include more than 50 scientific publications, the 1993 Sir Casimir Stanislaus Gzowski Medal from the Canadian Society for Civil Engineering, the 2002 Moisseiff Award from the American Society of Civil Engineers and the 2007 Excellence in Teaching and Research Award from BCIT.

Robert Tremblay, P.Eng., Ph.D., is an adjunct professor in the Department of Civil, Geological and Mining Engineering at Polytechnique Montréal in Montréal, Canada, where he held the Canada Research Chair in Earthquake Engineering from 2003 to 2017. He received his bachelor's (1978) and master's (1988) degrees from Université Laval in Québec City, Québec, and completed his Ph.D. in 1994 at the University of British Columbia in Vancouver, BC. Before undertaking his doctoral studies, Dr. Tremblay worked for 10 years in the industry. His research work concentrated on the seismic design and stability of steel structures. His most significant contributions include numerical and experimental investigations of the seismic response of steel braced frames, including advanced framing systems for enhanced seismic performance such as buckling-restrained, rocking and self-centring braced frames, as well as braced frames with elastic spines. He is the author of more than 500 technical articles and technical reports. He has taken part in several reconnaissance visits to inspect structural damage after major earthquake events. He is a former member of the Standing Committee on Earthquake Design of the National Building Code of Canada and the CSA S16 Technical Committee on Steel Structures. He also was a member of the American Institute of Steel Construction (AISC) Task Committee 9 on seismic design of steel structures from 2005 to 2021. He authored the chapter on seismic stability for the 2010 Guide to Stability Design Criteria for Metal Structures produced by the Structural Stability Research Council, an international body bringing together researchers and industry representatives. He received the P.L. Pratley Award in 1997, the

Sir Casimir Stanislaus Gzowski Medal from the Canadian Society for Civil Engineering in 2004 and 2007, the Raymond C. Reese Research Prize from the American Society of Civil Engineers in 2004, the AISC Special Achievement Award in 2009 and the AISC Lifetime Achievement Award in 2022. He was named a fellow of the Canadian Society for Civil Engineering in 2004 and a fellow of the Canadian Academy of Engineering in 2012.

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Chapter 1
Preliminary Notions

1.1 BRIEF HISTORY OF EARTHQUAKE ENGINEERING

Earthquake engineering is a relatively young discipline that grew out of the traditional fields of civil engineering and geophysics and saw major developments through the 20th century. The historical review of progress in earthquake engineering presented in this chapter draws significantly from a presentation by George Housner (1910–2008) (1984) at the Eighth World Conference on Earthquake Engineering held in San Francisco in 1984, as well as work by Elnashai (2002), Housner (2007) and Reitherman (2008).

The foundations of earthquake engineering were elaborated almost exclusively by British scientists in the 18th and 19th centuries. Although England has relatively low seismicity, the Industrial Revolution (1700–1900) spurred the growth of various sciences. Robert Hooke (1635–1703), known for his law of elasticity, was one of the first scientists to become interested in the earthquake phenomenon. In 1667 and 1668, Hooke delivered lectures on earthquakes and volcanoes at the Royal Society of London, the world's oldest scientific academy. Hooke's *Discourse of Earthquakes*, published in 1705, describes the early reasoning on the geological occurrence of earthquakes.

In the 19th century, no distinction was made between seismology and earthquake engineering. The term "seismology" is derived from the Greek word *sismo* (vibration) and was coined by British engineer Robert Mallet (1810–1881). Mallet also introduced several other terms to the vocabulary of earthquake engineering, including "epicentre" and "focal point." In 1848, he published the concept for the first electromagnetic seismograph, but the instrument was never built during his lifetime. Italian-born Luigi Palmieri (1867–1896) later modified Mallet's design, built the first seismograph and successfully recorded the first modern earthquake records.

Meanwhile, on the other side of the globe at the Imperial College of Engineering in Tokyo, John Milne (1850–1913), James Ewing (1855–1935) and Thomas Gray (1850–1908) founded the Seismological Society of Japan. As one of its first activities, the society funded the fabrication of seismographs to detect and measure ground motion during earthquakes. In 1880, it became the first scientific association entirely dedicated to the study of seismic phenomena, forerunner to the International Association for Earthquake Engineering,[1] which is still in existence today.

Three major earthquakes at the turn of the 20th century strongly contributed to the development of earthquake engineering: the 1891 earthquake in Mino-Owari (Japan), the 1906 earthquake in San Francisco (USA) and the 1908 earthquake in Reggio Messina (Italy). The Reggio Messina earthquake on December 28, 1908, is particularly important for the devastation it caused—more than 83,000 people died—and also because it triggered development of the first methods of earthquake-resistant construction and the birth of modern earthquake engineering. The Italian government set up a special committee of nine practising engineers and five engineering professors mandated to study the earthquake and formulate recommendations. In their report, Modesto Panetti (1875–1957), a professor of applied mechanics at the University of Turin, proposed that for active seismic zones, civil engineering structures should be calculated with a uniform static lateral load represented by a "seismic coefficient" expressed as a fraction of the structure's weight. He also emphasized that the effects of earthquakes on structures are in fact a problem of "structural dynamics." This problem, he wrote, was "much too complicated" to address, and a simpler static approach was preferred for the design of earthquake-resistant structures. This approach remained in use until engineers finally gained access to powerful computers much later in the 20th century. In 1909, Arturo Danusso (1880–1968), a professor of structural engineering at the University of Milan, published a paper that fully explained the application of the static approach proposed by Panetti and included a novel method for the dynamic analysis of buildings (Sorrentino, 2007). The static method was slowly integrated into building codes and remained pretty much unchanged until the 1940s. After Tokyo's great earthquake in 1923, Japanese seismic provisions adopted a seismic coefficient of 10%. The city of Los Angeles, on the other hand, adopted a seismic coefficient of 8% following the 1933 Long Beach earthquake and, in 1943, adopted a seismic coefficient that varies with the height of the building. These events marked the birth of the static approach method, which is still found today in most modern building codes around the world.

1.2 DEVELOPMENT OF EARTHQUAKE ENGINEERING IN THE UNITED STATES

Between the years 2000 and 2012, more than 800 earthquakes with Richter magnitudes larger than 5.0 occurred each year in the United States (US).[2] (The formal definition of the logarithmic Richter magnitude is presented in Chapter 2 of this textbook.) Both Alaska, located on a subduction plate boundary, and California, situated on the boundary of the North American and Pacific tectonic plates, have more earthquakes per year than the combined total of the rest of the United States. The March 27, 1964 "Good Friday" earthquake in Alaska is the largest earthquake (magnitude of 9.2) ever recorded in the United States.

1 http://www.iaee.or.jp/

2 https://www.usgs.gov/programs/earthquake-hazards/lists-maps-and-statistics

Chapter 2

Elements of Seismology and Seismicity

2.1 INTRODUCTION

The earthquake design of a structure is dictated by the degree of seismic activity at the construction site. Many seismological factors directly influence the work of structural engineers. These include:

- The distribution of earthquake sources affecting the construction site
- The fault mechanisms of the various sources
- The seismic activity of the various sources in terms of the recurrence of magnitudes
- The ground-motion intensity
- The attenuation of the ground motion with distance from the source to the site
- The geological, geotechnical and topographical conditions at the site

This chapter presents an overview of the fundamental properties of these various seismological aspects to acquaint structural engineers with the "language of seismologists."

2.2 CAUSES OF EARTHQUAKES

2.2.1 Natural earthquakes

Most natural earthquakes occur in Earth's crust in two specific zones on the planet (Fig. 2.1): the Circum-Pacific Belt, which includes South America, the California coast, Alaska, Japan, Taiwan, the Philippines and New Zealand; and the Alpide Belt, which includes the Mediterranean, North India and Indonesia. (See Sect. 2.3 for more information on how earthquakes arise.)

Damage from an earthquake can be caused by:

- Fault movement (one-dimensional)
- Ground shaking (three-dimensional)

- Liquefaction or soil failure (Chap. 5)
- Tsunami (sea) or seiche (lake)
- Flooding
- Fire

Most structural damage is caused by ground shaking.

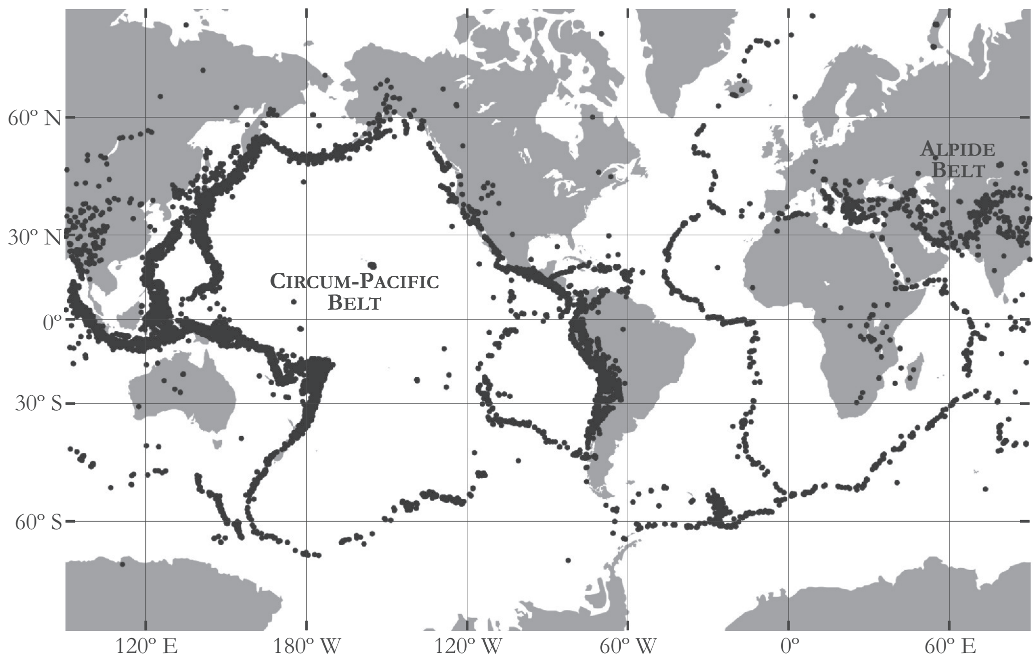

Figure 2.1 World seismicity from 1977 to 1993, magnitude > 5.5

2.2.2 Induced earthquakes

Some human activities can influence the amplitude and distribution of strains in Earth's crust. Interventions such as filling water reservoirs, mining, excavating large quarries, injecting fluids under high pressure to generate geothermic energy, digging oil wells and setting off underground nuclear explosions can cause major induced earthquakes.

Among these human activities, the filling of water reservoirs usually causes the most severe induced earthquakes. Reservoir-induced seismicity has been observed at over 70 locations worldwide and has caused earthquakes reaching magnitudes of 6 on the Richter scale (Sect. 2.10). For further reading, Tiwari *et al.* (2017) is an excellent reference on earthquakes induced by the filling of water reservoirs. Only naturally occurring earthquakes are considered in this textbook.

Chapter 3

Elements of Seismic Hazard Analysis

3.1 DEFINITION OF A DESIGN EARTHQUAKE

As seen in Chapter 2, a region's seismic risk depends on the probability of an earthquake occurring and causing a certain level of ground shaking (seismic hazard), and on the structural vulnerability of the built structures on that ground. An important step in designing a structure in a seismic zone is defining a design earthquake for that specific location. The design earthquake is defined as "the ground movement that will be used in the calculation of the earthquake-resistant design" (EERI Committee on Seismic Risk, 1984). Three parameters are used to define the design earthquake: the level of seismic activity in the region, the specific conditions (geotechnical or geological) at the site and the acceptable risk level. The third criterion, acceptable risk level, is subjective and is defined as "a probability of social and economic consequences due to earthquakes that is low enough to be judged by appropriate authorities to represent a realistic basis for determining design requirements for engineered structures" (EERI Committee on Seismic Risk, 1984).

The probabilistic method, originally developed by Cornell (1968), is used to determine the design earthquake for all locations in Canada and the United States. This method gives a relative quantification of the design earthquake for a given site in terms of seismic parameters (peak ground acceleration, spectral acceleration at various periods, etc.), and considers uncertainties associated with the occurrence and effects of important earthquakes.

Probabilistic seismic hazard analysis cannot predict the date, magnitude, location or effects of future earthquakes in active seismic zones. However, it provides a basis for the seismic design of structures at a given location.

The probabilistic method was used to construct the seismic zoning maps that are included in the National Building Code of Canada (NBCC) and in the International Building Code

(IBC) in the United States. It combines two separate models: a seismicity model and an attenuation model. The seismicity model describes the geographical distribution of potential active source zones (seismotectonic sources) and the probabilistic distribution of magnitudes in each source. The attenuation model describes the effect of an earthquake originating from a specific seismotectonic source, at any given site, as a function of the magnitude and the source-to-site distance.

3.2 SEISMICITY MODEL

3.2.1 Description

On any given fault (source) within a region, earthquakes occur at irregular intervals. Seismologists have long searched for meaningful patterns of earthquake occurrence. The longer the historical record, the more likely it is that patterns will emerge. In most regions, the useful historical record is short, dating back only a few decades, or sometimes one or two centuries. The great exceptions are China and the eastern Mediterranean, which have useful records dating back over 2000 years.

Over any given time period, the historical distribution of events along a fault line can be represented by an empirical recurrence relation proposed by Gutenberg and Richter (1965):

$$\log N = A - bM \tag{3.1}$$

where log = logarithm (log = ln or $\log_{10}$)
N = number of earthquakes of magnitude greater or equal to M per unit of time and unit of area (also called the return frequency or recurrence rate); note that $1/N$ represents the return period (in units of time) of an earthquake of magnitude greater or equal to M over a unit area
M = magnitude (M_L, m_b, M_s, M_w or m_N)
A, b = seismic constants for the region

Table 3.1 shows typical values of the seismic constants A and b for different regions of the world. These values, presented for comparison purposes only, have been obtained from very large areas. Usually, the seismic constants are defined for much smaller seismotectonic sources. When seismic data exist for a given region, a plot of M against log N can be made and a linear regression analysis is used to estimate the seismic constants A and b. Figure 3.1 illustrates this type of calculation. Table 3.2 compares the recurrence rates and return periods of earthquakes of magnitudes greater than or equal to 5, 6 and 7 for the western and eastern United States based on the seismic constants given in Table 3.1. The higher seismic activity in the western United States is clear from this data.

Chapter 4

Elements of Structural Dynamic Analysis

4.1 INTRODUCTION

A dynamic analysis is required to more accurately evaluate the response of a structure during an earthquake. Structural engineers are familiar with the static analysis of structures, for which a single load pattern is applied and a time-independent solution is obtained. A dynamic analysis, on the other hand, generates a time-varying solution. By studying this solution, a structural engineer can determine the maximum deformations and forces that the structure will sustain during this loading and can then design for them. The main purpose of this chapter is to present the fundamental notions of structural dynamics and to introduce analysis methods that will enable the reader to calculate the response of structures subjected to various types of dynamic loads, with a specific emphasis on earthquake ground motions.

Almost all structures are subjected to dynamic loads during their life span. Although most loads are in fact applied dynamically to structures, when the rate of the loading is slow or when the load is sustained over a long period of time, the dynamic effects on the structure are very small and therefore such loads can be considered as static. When the loads are applied quickly or when the loading varies over a period of time that is close to the natural period of the system (a concept discussed in detail in this chapter), the structure's dynamic response is influenced by the time-varying properties of the loading. Figure 4.1 illustrates the effect of a time-varying load $F(t)$ that is applied to a simple cantilever structure. When the load is applied slowly, the structure deflects to a static equilibrium position Δ_{st}. When the same magnitude of loading is applied to the structure in a time-varying pattern, the structure's response is amplified and reaches a maximum of $\Delta_{\max}$, which can be significantly greater than Δ_{st}.

The main purpose of dynamic analysis is to evaluate the time variations of stresses and deformations in structures that are caused by arbitrary dynamic loads. Often, once the dynamic response of a structure is assessed, an equivalent static loading pattern, which represents the maximum loading state that occurs during the structure's dynamic response, is defined and used in the design of the structure. For the example illustrated in Figure 4.1, this equivalent static force would be given by $F_{es} = k\Delta_{max}$, where k is the lateral stiffness of the system.

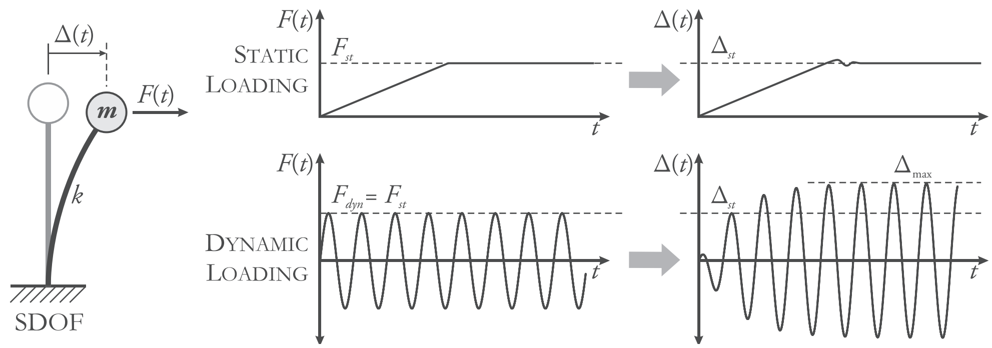

Figure 4.1 Response of a simple structure to a static load and to a dynamic load of the same magnitude

4.1.1 Examples of dynamic loads

As illustrated in Figure 4.2, some of the main dynamic loads that are of interest to structural engineers include floor vibrations caused by the movement of occupants, vibrations caused by rotating machinery, wind loading, blasts or impacts, and earthquake loading, which is imparted to a structure as a result of ground motion.

When the properties of dynamic loads are not defined at all times, but are characterized by statistical parameters such as the mean and standard deviation of the amplitude of the loading and the frequency content, they are referred to as random loads. The most common types of random loads that are encountered by structural engineers are wind pressures and floor vibration loads. The seismic loads that are used for the design of structures are also random loads, as they are defined using a highly uncertain probabilistic seismic hazard analysis procedure (as discussed in Chapter 3) and are intended to represent the characteristics of a number of possible earthquake scenarios that could impact the structure at the design site.

When the amplitude, direction and location of loading are known at all times, it is referred to as a deterministic load. Historical earthquakes that have been recorded at a given site and vibrations caused by machinery that is rotating at a specific frequency are two examples of deterministic dynamic loads. Deterministic dynamic loads can be of a periodic (or cyclic) nature when the loading pattern is repeated and exhibits the same time-varying properties for many continuous cycles. Nonperiodic loads can be very short, as is the case with blast or impulse loads, or of longer duration but without any repetition in the loading pattern.

This chapter focuses on the dynamic response of structures that are subjected to deterministic loads, both periodic and nonperiodic. Results presented herein for deterministic loading conditions are also extendable to the response of structures to random type loadings.

Chapter 5
Elements of Soil Dynamics

5.1 INTRODUCTION

The seismic hazard maps for Canada and the United States have been developed for specific local soil conditions at each respective site (Chap. 3). In this chapter, we look at a structure situated on a soil deposit, as illustrated in Figure 5.1, and investigate the following questions: Does a soil deposit influence the ground motion at the base of a structure? How is the dynamic response of a structure affected by the presence of a soil deposit?

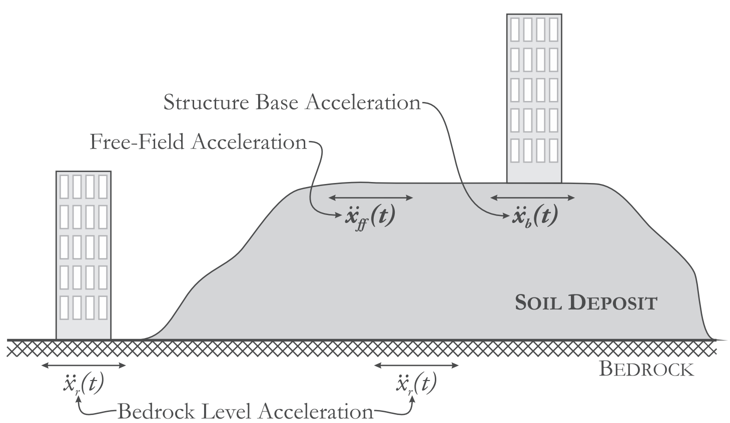

Figure 5.1 Effect of a soil deposit during an earthquake

Four problems can be foreseen for structures that rest on a soil deposit:

1. The seismic loads on the structure may be greater than for the same structure standing on bedrock (soil amplification).
2. During violent vibrations, the soil may lose its load-bearing capacity completely and move from a solid to a liquefied state (soil liquefaction).

3. Following vibrations of large amplitude, the soil may collapse and cause differential settlement of the structure's foundation (soil spreading).
4. The soil may continue to settle after the end of an earthquake (post-shaking settlement).

In order to better understand the causes of these problems, we will first consider the dynamic behaviour of a soil deposit during an earthquake.

5.2 SOIL DYNAMIC RESPONSE: LUMPED-MASS ANALYSIS OF HORIZONTALLY LAYERED SOIL DEPOSIT ON RIGID BEDROCK

5.2.1 Description

In many cases, the surface motion of a soil deposit originates mainly from the vertical propagation of shear waves (S-waves) from the bedrock to the ground surface. In general, a soil deposit is made of many layers (clay, silt, sand and so on), each of which exhibits different mechanical properties. If the rock layer or the different soil layers are on an incline, finite-element techniques must be used to compute the dynamic response (Kramer, 1996). However, if the rock or the interface between the different soil layers is essentially horizontal, each layer can be considered linear and elastic, and shear waves propagate in the vertical direction from bottom ($z=-h$) to top ($z=0$). The horizontal displacement at any point in the soil and at any given time, $u(z, t)$, is governed by the one-dimensional wave equation (Sect. 2.7.4):

$$\frac{G}{\rho}\frac{\partial^2 u}{\partial z^2}=\frac{\partial^2 u}{\partial t^2} \tag{5.1}$$

where G is the shear modulus of the soil and ρ its density. Although Equation 5.1 can be solved for simple shear stress wave profiles, numerical solutions are usually required when earthquake excitation is considered. This chapter builds on the structural dynamic concepts presented in the previous chapter to develop a simple lumped-mass model to analyze the dynamic response of a soil deposit during an earthquake. A simple mechanical model of a soil deposit (Fig. 5.2) is presented as a semi-infinite medium with a unit width and an infinite length. The mass of the soil is assumed to be concentrated equally over and under each layer.

$$\begin{aligned} m_1 &= \frac{\rho_1 h_1}{2} \\ m_i &= \frac{\left(\rho_{i-1}h_{i-1}+\rho_i h_i\right)}{2} \quad i=2,3,\ldots N \end{aligned} \tag{5.2}$$

Chapter 6

Elements of Seismic Design Procedures for Building Structures

6.1 INTRODUCTION

A number of seismic design approaches have been proposed since the early days of earthquake engineering. Although in principle, the seismic design of structures could rely on an elastic dynamic analysis (see Chapter 4) to determine the force demands on all structural members and then size the members to resist these loads in the elastic range, such a design approach would be cost-prohibitive. This approach could also be potentially dangerous and lead to unsatisfactory seismic performance given the great uncertainty that persists in defining the design ground motion: there is a real possibility that a structure will be subjected to a ground motion greater than the one considered in the design. The prediction of ground motion effects on the structure is also uncertain. In particular, soil-structure interaction may significantly affect the demand imposed on the structure, and the structural response may be affected by nonstructural components. In practice, a perfectly accurate prediction of these effects is still not possible, even when a detailed nonlinear dynamic analysis is performed. The force demands on the seismic force–resisting system (SFRS) predicted by the analysis may therefore be exceeded during a severe earthquake, which could lead to catastrophic failures and even the structure's collapse if brittle SFRS components are overloaded. For this reason, modern seismic design codes have adopted a different design philosophy that takes advantage of the ductility of structures. Explicitly chosen and carefully detailed elements are allowed to yield and undergo inelastic deformations while limiting the forces that must be resisted during the earthquake. All other members in the structure are designed to maintain their integrity during ground shaking while the yielding elements undergo inelastic deformations, thus safeguarding the integrity of the structure and protecting it against the possibility of collapse.

This chapter explains the concept of ductile response that is at the base of the seismic design philosophy in modern seismic design codes. It then outlines the application of this design philosophy to a number of SFRSs that are commonly used. The methods presented are referred to as force-based seismic design, since the approach is centred on defining seismic design forces and providing the SFRS components with sufficient strength to resist them. Design codes and material standards include special design and detailing requirements to ensure that the SFRS will have sufficient deformation capacity to safely withstand the expected seismic deformations. These requirements are introduced in the examples presented at the end of the chapter (Sects. 6.4.11 and 6.5) and are discussed in detail in Chapters 7, 8 and 9 for steel, concrete and wood structures, respectively. Design seismic loads are determined from estimates of the elastic force demand on the structure. These forces are then reduced in view of the structure's capacity to undergo inelastic deformations in case of an extreme but rare event. Considerable research has been devoted to the development and refinement of these force-based methods. Several SFRSs have been developed, along with detailing requirements that must be satisfied to enhance inelastic response. Current building codes incorporate the findings of this research.

More recently, displacement-based design methods and, more specifically, direct-displacement-based design has been developed as an alternative to force-based methodologies (Priestley *et al.*, 2008). While pursuing the same basic aim of designing structures to exhibit a controlled ductile seismic response, its main premise is that damage is related to deformations in the structure, and it therefore seeks to achieve structures that meet a predefined deformation state during an earthquake. In this methodology, seismic design forces are derived based on an explicit consideration of the nonlinear force deformation characteristics of the structure as well as its ability to dissipate seismic energy as it undergoes repeated cyclic deformations. A number of studies have shown that this methodology allows for a more rational seismic design and addresses some of the inherent weaknesses of force-based methods. Although this methodology is not presented here, as it is not currently part of any major seismic design codes, the concepts outlined in this chapter and the general characteristics of the SFRSs presented are directly applicable to any ductile design methodology. The reader is referred to Priestley *et al.* (2008) for a detailed explanation of this method and its application to the seismic design of building structures.

Most current seismic design codes aim to achieve the life-safety and collapse prevention performance levels that ensure that there is no loss of life during a major earthquake as a result of structural collapse. It has long been recognized, however, that the impact of an earthquake must also consider the socio-economic consequences of loss of property and loss of operations. As such, a more comprehensive approach, referred to as performance-based earthquake engineering, has been developed over the last few decades (SEAOC, 1995; ASCE, 2000; ATC, 2006). When adopting this more comprehensive design methodology, it is possible to characterize the performance of a structure under multiple seismic hazard levels, such as frequent earthquakes, design level earthquakes and maximum credible earthquakes, and to assess the expected damage and economic losses associated with each of these hazard levels. Damage is assessed based not only on the structural system but also by considering the nonstructural components, the building contents, the cost of repairs and the expected downtime for operations housed in the structure. The methodology is formulated in a probabilistic framework and therefore allows the owner of a structure to associate the cost of the structure (or of the repair of an existing structure) directly with

Chapter 7

Elements of Seismic Design and Detailing for Steel Buildings

7.1 INTRODUCTION

This chapter provides an introduction to the basic principles and general requirements governing the design of steel seismic force–resisting systems (SFRS), including concentrically braced frames (CBFs), buckling restrained braced frames (BRBFs), eccentrically braced frames (EBFs), moment resisting frames (MRFs), and plate walls (PWs) or plate shear walls (PSWs). Detailed explanations and examples are provided for two of these framing systems: CBFs in Section 7.3 and MRFs in Section 7.6. The examples presented for these systems complement the building example used in Section 6.5 to illustrate the seismic provisions of building codes. An introduction to the basic seismic design requirements applicable to BRBFs and EBFs is also presented in Sections 7.4 and 7.5. Section 7.2 provides an overview of the general seismic provisions that apply to all SFRSs.

The chapter's focus is on the most ductile system categories, as these generally represent the most appropriate solution for moderate and high seismic applications. As discussed in Chapter 6, these ductile systems can be designed for lower seismic loads and are expected to offer a more robust inelastic seismic response.

Other structural steel systems can be designed and built from steel to resist the actions of earthquakes. For instance, cantilevered column systems consisting of fixed-base columns supporting a roof or canopy are commonly used to resist lateral loads in flexure. AISC 341-22 also contains provisions for the steel truss MRF system (system C2 in Table 7.3) and for several steel-concrete composite systems (not shown in Table 7.3). Dual systems are also defined in ASCE 7-22 (systems D and E in Table 7.3) but are not examined here.

The chapter only covers the main seismic design requirements applicable to the systems described. Several other design and detailing rules, as well as material and fabrication requirements, may also apply to these systems, and the reader must consult the design standard applicable to actual design situations. The reader will also find very useful additional information on the seismic design of steel SFRS in the commentary documents that accompany

CSA S16:19 and AISC 341-22. AISC (2024) contains several detailed examples of seismic design for steel SFRSs, including connection design and detailing. Bruneau *et al.* (2011) provides background information on the ductile behaviour and design of steel structures, including their seismic design. Specific references on each seismic force–resisting system are given in the corresponding chapter sections. Sabelli and Bruneau (2007) present information on the seismic design of steel PWs.

7.1.1 Ductile vs nonductile failure modes

Typical stress-strain response for steel under axial tension is shown in Figure 7.1a. The initial linear elastic behaviour is characterized by a high stiffness described by Young's modulus, $E = 200\,000$ MPa. This elastic range is followed by a plastic region where the stress remains nearly constant, at a value referred to as the steel yield stress, F_y. The yield plateau extends from the strain at yield, $\varepsilon_y = F_y/E$, to the strain at onset of strain hardening, ε_{sh}. In the strain hardening range, the stress increases up to the tensile strength, F_u. In a tensile test, necking of the coupon begins at this point and the apparent applied stress decreases as the cross-section progressively reduces until the sample ruptures at the rupture strain, ε_r.

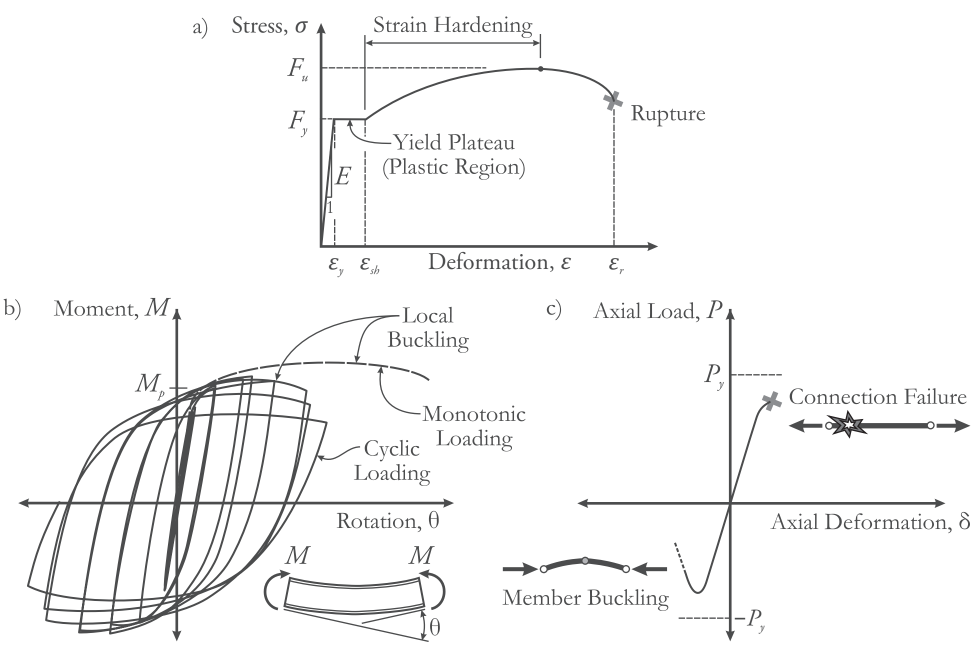

Figure 7.1 a) Stress-strain response of steel; b) inelastic response of flexural member under monotonic and cyclic responses; c) connection failure

Steel offers high inherent ductility that can be used to resist earthquake effects through ductile response in the inelastic range. Steel structural members subjected to inelastic axial deformations in tension are expected to exhibit the same behaviour as that obtained from tensile coupon tests shown in Figure 7.1a. In Figure 7.1b, steel members subjected to monotonically increasing flexural demand can develop their full plastic moment capacity, M_p, and

Chapter 8

Elements of Seismic Design and Detailing for Reinforced Concrete Buildings

8.1 INTRODUCTION

Seismic force–resisting systems using reinforced concrete construction have played a significant role in advancing the understanding of structural response during earthquakes and enabling the design of life-safe structures.

Historically, moment-resisting frames (MRF) have been the most common form of reinforced concrete seismic force–resisting systems as the first attempts to incorporate such systems used typical vertical load–resisting moment-resisting frames to sustain lateral loading. Through extensive research and observation of their behaviour during earthquakes, the design and detailing of these systems for earthquake-induced forces and deformations has progressed to a point that MRFs and special moment-resisting frames (SMRF) exhibit excellent ductile nonlinear behaviour.

More recently, reinforced concrete seismic-resisting walls, often referred to as shear walls, have become the prevalent method of construction in high-seismicity regions. Shear walls are used in a wide variety of situations and can satisfy not only seismic requirements but also functional needs such as fire and acoustic insulation. For mid- and high-rise buildings, reinforced concrete walls are often located around the elevator and mechanical service shafts, thus forming a core to the building that has high flexural and torsional stiffness (much like a closed tube). When well designed, these structural walls exhibit predominantly flexural behaviour and are considered analogous to a vertical cantilever beam.

When structural walls are used in low- to mid-rise construction, they often have a low height-to-length (h_w/l_w) ratio. As the wall aspect ratio decreases, the behaviour of a wall shifts from flexure- to shear-dominated and the walls are considered to be "squat walls." In the Canadian code for the design of concrete structures, CSA A23.3:19 (CSA, 2019a), the limiting aspect ratio between flexural and squat walls (and the different design requirements that apply to each type) is $h_w/l_w \leq 2.0$, while the American concrete building design code, ACI 318-19 (ACI, 2019a), sets this ratio at 2.0. In this chapter, only flexure-dominated walls are considered (i.e., those with an aspect ratio greater than 2.0).

A third reinforced concrete structural system used for seismic force resistance is a coupled wall. Coupled walls tend to combine short walls (i.e., the cross-section length of the wall is small) that lie on the same plane and relatively short beams that span between the two adjacent walls. These beams will often have a ratio of the clear span length to the beam depth (l_u/h_b) of 2 or less and therefore add a significant amount of stiffness to the system. As shown in the design examples in this chapter, coupled walls are often found perpendicular to structural walls that form the seismic force–resisting core in modern buildings.

This chapter references the 2019 edition of the Canadian code for the design of concrete structures, CSA A23.3:19, as this is the version currently cited by the most recent edition of the National Building Code of Canada (NBCC, 2020).

8.2 BASIC PHILOSOPHY OF REINFORCED CONCRETE SEISMIC DESIGN

As discussed in Chapter 6, the application of capacity design principles (Park and Paulay, 1975) is paramount to designing a seismic force–resisting system. The Canadian concrete code, CSA A23.3:19, explicitly enforces the use of capacity design for buildings using $R_d \geq 1.5$. The American concrete design code, ACI 318-19, contains clauses for reinforced concrete seismic systems that inform a basic level of capacity design. However, as shown in examples later in this chapter, the two codes are different and will not necessarily result in similar designs.

In MRF and SMRF systems, a weak-beam, strong-column ductile mechanism is typically assumed as the inelastic mechanism. Columns that form plastic hinges at their base are also allowed to complete the mechanism. The strength hierarchy stipulated in the Canadian and American concrete codes is as follows:

- Beam shear capacity exceeds the shear demand associated with the nominal beam moment capacity being developed simultaneously at each end of the beam.
- The total column moment capacity framing into the beam-column joint exceeds the sum of the nominal beam moments framing into the beam-column joint.
- The column shear capacity exceeds the shear associated with the nominal beam moments framing into the column and the column moment demands.
- The beam-column joint shear capacity exceeds the sum of the shears associated with the nominal beam and associated column moments.
- The foundation strength must be greater than the factored column moment and axial load combination.

The significant differences between the two codes are highlighted in the design examples presented in this chapter.

Chapter 9

Elements of Seismic Design and Detailing of Wood Buildings

9.1 INTRODUCTION

Wood is a complex building material, being multicomponent, hygroscopic, anisotropic, inhomogeneous, discontinuous, inelastic, fibrous, porous and biodegradable (Bodig and Jayne, 1982). On the microscopic scale, wood is made of long, parallel, hollow cells that are primarily made of cellulose fibres, a natural polymer. These cells are bound together within a matrix of lignin. Lignin is another, weaker natural polymer that acts as a glue, holding the cells together. The cells are all oriented in the same direction aligned with the tree's growth direction. This gives wood higher strength in the parallel-to-grain direction and lower strength in the perpendicular-to-grain direction. This microscopic arrangement also causes wood to be a relatively brittle material, especially for failures in shear and tension, characterized by the failure at the weak lignin interfaces between cells. In compression and bearing, wood material exhibits progressive crushing that is not sudden but is permanent and unrecoverable.

Due to the brittle mechanical behaviour of wood, steel connections or fasteners are necessary for earthquake applications to provide ductility and energy dissipation for high-performance structures made of wood. In these structures, it is typically the steel fasteners that provide yielding in the connections through ductile bending of the fasteners. Without the ductility provided by steel connections or fasteners, wood structures must essentially be designed to remain linear during an earthquake.

In this chapter, two types of earthquake-resistant wood structures will be described: light-frame wood buildings and cross-laminated timber (CLT) wood buildings. Light-frame wood buildings have been common in North America for many decades; they use shear walls consisting of lumber studs and wood sheathing. The nails connecting the sheathing to the studs yield to provide the necessary ductility for earthquake resistance. In Canada, light-frame construction is limited to six storeys (NRC, 2022). In the US, light-frame construction is limited to three or four storeys depending on the occupancy (ICC, 2020). CLT construction, first developed in Europe in the 1990s, permits the design and construction of taller, earthquake-resistant wood buildings. In Canada, with the release of the NBCC 2020 (NRC, 2022), CLT buildings can be built as tall as 12 storeys using encapsulated construction techniques, where the CLT panels are fire-protected by covering them with gypsum board. In the US, CLT buildings can be built as tall as 18 storeys using encapsulated construction, or nine storeys with exposed wood (ICC, 2020).

9.2 LIGHT-FRAME WOOD BUILDINGS

Light-frame wood buildings constitute, by far, the greatest proportion of North America's building inventory. This is because most single-family residential buildings are light-frame wood buildings. In addition, light-frame wood construction is often an economical choice for low-rise apartments, condominiums and small commercial buildings.

In light-frame wood buildings, the walls, floors and roof elements are made of closely spaced, parallel, sawn-lumber framing members that are attached to wood panels made of plywood, oriented strand board (OSB) or gypsum wallboard. The framing members and panels are connected using mechanical fasteners, typically nails or screws.

The seismic performance of light-frame wood buildings in North America has generally been good given the built-in redundancy in the structural system, the relatively light building mass and the energy dissipation capacity of the ductile mechanical fastener connections. Wood-frame buildings rarely collapse during earthquakes. Wood buildings that have collapsed include those with improper anchorage on hillsides and those with soft storeys resulting from unsheathed cripple walls and tuck-under parking. These structural deficiencies were exposed during the 1994 Northridge earthquake in California. During this seismic event, although they generally did not collapse, many light-frame wood buildings experienced both structural and nonstructural damage. Since so many of the buildings were built using light-frame construction, approximately half of the $48 billion in losses resulting from the Northridge earthquake was attributed to damage sustained by wood-frame buildings (Kircher *et al.*, 1997).

This section provides an overview of the general seismic design and detailing requirements for light-frame wood buildings. It presents the specific seismic design provisions in Canada and the United States and applies these to several design and detailing examples. The Canadian design standard is the Canadian Standards Association's CSA O86:19 Engineering Design in Wood (CSA, 2019d). This standard is based on limit-states design. The United States design standards are the American Wood Council's 2018 National Design Specification for Wood Construction, abbreviated NDS (AWC, 2018a), and the 2021 Special Design Provisions for Wind and Seismic, abbreviated SDPWS (AWC, 2020). Both of these design documents provide for allowable stress design (ASD) and load and resistance factored design (LRFD) methods. Throughout this chapter, all references to CSA O86, NDS and SDPWS are based on the 2019, 2018 and 2021 editions, respectively. To supplement the Canadian design standard, the Canadian Wood Council publishes a wood design manual

Chapter 10

Elements of Seismic Design of Nonstructural Building Components

10.1 INTRODUCTION

Nonstructural components and systems in a building are not part of the structural load-bearing system, but they are subjected to the same dynamic environment experienced by the building during an earthquake. Architectural components, machinery and electrical and mechanical equipment mounted within buildings must be designed to withstand the forces and displacements that arise from the seismic response of the structure. Elevators and their counterweights, for example, are vulnerable to large structural displacements as well as to lateral forces. The mounting and support of motors, fans and other machinery and equipment need sufficient strength to resist the seismic forces transmitted through these components. Also, the failures of interior partitions, finishes and suspended ceilings pose hazards to occupants.

In many strong earthquakes that have struck North America in the twentieth century, as well as ones in our current century—such as the 2001 earthquake in Nisqually, Seattle (Filiatrault *et al*., 2001), the 2006 earthquake in Kona, Hawaii (Chock *et al*., 2006; Gupta and McDonald, 2008) and the 2010 Chile earthquake (Miranda *et al*., 2012)—losses from damage to nonstructural components has far exceeded losses from structural damage in most affected buildings. Nonstructural damage can severely limit the functionality of critical facilities, such as hospitals, as demonstrated in the 1994 Northridge earthquake (McGavin and Patrucco, 1994), the 2001 El Salvador earthquake (Boroschek and Retamales, 2001), the 2006 earthquake in Kona, Hawaii (Chock *et al*., 2006) and the 2010 Chile earthquake (Miranda *et al*., 2012). The harmonization of seismic performance between structural and nonstructural components is vital. Even if the structural components of a building perform well enough that people can occupy it immediately after a seismic event, failure of architectural, mechanical or electrical components and contents can lower the performance level and functionality of the entire building system. The failure of nonstructural building components can become a safety hazard or can hamper the safe movement of occupants as they evacuate or of rescuers as they enter the building (Villaverde, 2004).

Nonstructural components represent the majority of the total investment in typical buildings. According to Miranda and Taghavi (2003), nonstructural components compose approximately 82%, 87% and 92% of the total monetary investment in office, hotel and hospital buildings in the United States, respectively (Fig. 10.1). Clearly, the investment in nonstructural components is far greater than that allotted to structural components and framing. Furthermore, damage to nonstructural components occurs at seismic intensities much lower than those required to produce structural damage (Miranda and Taghavi, 2003). Therefore, it is not surprising that in many past earthquakes, losses from damage to nonstructural building components have exceeded losses from structural damage.

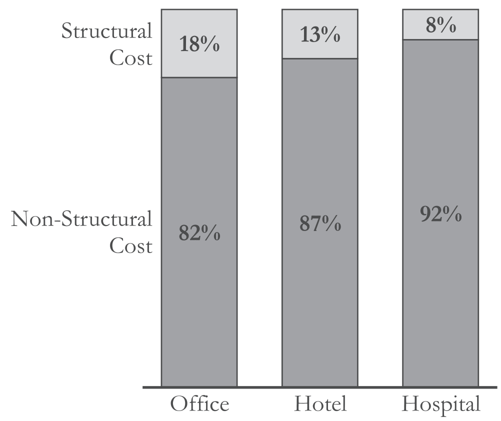

Figure 10.1 Relative investments in typical buildings (after Miranda and Taghavi, 2003)

In comparison to structural components and systems, much less specific guidance is available on the seismic design of nonstructural building components for multiple performance levels. Limited basic research has been done in this area, so design engineers are often forced to start almost from square one: observe what goes wrong and try to prevent it from happening again. This is a consequence of the empirical nature of current seismic regulations and guidelines for nonstructural components. The design information currently available in codes, standards and guidelines is based on judgment and intuition for the most part rather than on experimental and analytical results. Summaries of many important aspects of the seismic behaviour of nonstructural components, as well as of the evolution of research and code efforts in the last thirty years, can be found in Soong (1995), Filiatrault and Christopoulos (2002) and Filiatrault and Sullivan (2014).

This chapter first classifies and reviews the performance of nonstructural building components during recent earthquakes in North America to highlight the importance of considering the seismic design of nonstructural components. It then presents the current regulations and guidelines for the seismic design and specifications of nonstructural building components in Canada and the United States. Finally, we review the seismic qualification requirements for important nonstructural building components that have been introduced recently in

Chapter 11

Introduction to Supplemental Damping and Seismic Isolation Systems

11.1 INTRODUCTION

Previous chapters have covered the principles of conventional earthquake-resistant design, which ensure an acceptable safety level while preventing catastrophic failures and loss of life under the design-level seismic hazard. When a structure does not collapse during a major earthquake, and occupants can evacuate safely, the structure is considered to have fulfilled its purpose, even if it is never functional again and may ultimately have to be demolished. However, this represents a considerable cost to building owners, creates significant challenges for the post-earthquake emergency response and can have a potentially debilitating long-term impact on the city's, region's or even country's economy when a large number of structures in that same area are deemed to be total losses following an earthquake.

Designing for the so-called life-safety performance level is considered adequate for ordinary structures and has been the basis of modern seismic provisions. The design process does not currently account for the cost of damage to equipment and stored goods or the cost associated with the loss of operation following a moderate or strong earthquake. Although this current seismic design approach remains the general goal of any structure engineered in an earthquake-prone area, socio-economic realities are pushing the bar considerably higher. Critical mission-control facilities and structures such as hospitals, police stations, communication centres and major bridges must be designed to achieve significantly higher performance levels under severe earthquake shaking because of their importance in the immediate emergency response following a catastrophic event.

Furthermore, building owners and insurance companies are increasingly considering the impact of a major earthquake on their portfolios as a factor in their economic decisions. The cost of a new structure designed to meet higher performance levels or the cost of an upgrade to an existing structure are weighed against the estimated losses associated with damage, loss of property and downtime in the event of a major earthquake.

Over the last 30 years, a large body of research has been devoted to developing innovative earthquake-resistant systems to raise the seismic performance level of structures while keeping construction costs reasonable. Most of these systems are intended to dissipate the seismic energy introduced into the structure during an earthquake through supplemental damping mechanisms, or to protect the main structural elements from receiving this energy using isolation systems.

Supplemental damping and seismic isolation systems are now considered mature technologies and are used to achieve higher performance levels at a reasonable cost. This design approach aims to minimize damage to structural and nonstructural elements even under severe earthquake ground motions. This chapter provides a brief overview of common supplemental damping and seismic isolation systems that have demonstrated their considerable value through analytical studies, experimental testing and actual structural implementations. The discussion, inspired by the textbook by Christopoulos and Filiatrault (2022), focuses on passive energy dissipation systems and seismic isolation systems.

Before presenting the various damping and isolation systems, this section provides a brief look at the energy balance formulation for structural systems subjected to earthquake ground motions. This enables a more intuitive understanding of the effects of energy dissipation and seismic isolation on improving structural response during ground shaking.

11.2 RAIN FLOW ANALOGY

Because supplemental damping and seismic isolation systems alter the migration of energy quantities in a structure during an earthquake, it is important to first visualize the flow of various quantities of energy in a conventionally designed structural system as it dynamically responds to ground shaking. The rain flow analogy (Christopoulos and Filiatrault, 2006, 2022) shown in Figure 11.1 is helpful in this exercise. This figure illustrates a fictitious hangar with a retractable roof subjected to a rainstorm (Fig. 11.1a). The rainstorm represents the earthquake input, while the amount of rainwater entering the system represents the total seismic energy input into the structure. The amount of rainwater entering the system depends on the size of the roof opening, which symbolizes the dependence of the input energy on the structural properties during an earthquake and emphasizes that the input energy is not the same for every structure subjected to the same ground motion. If the roof is completely open, the structure will absorb all the seismic energy input generated at the site by the earthquake. In this case, there is a quasi-resonance between the ground motion and the dynamic response of the structure.

If the roof is completely closed, the structure does not receive any seismic input energy. This situation corresponds to a perfectly isolated structure. The amount of rainwater entering the structure, V_{in}, symbolizing the seismic input energy, is collected just below the ceiling line and is routed toward a kinetic energy pail. The amount of rainwater collected by this pail represents the kinetic energy generated by the masses of the structure as their inertia reacts to the seismic input energy transmitted to the structure. The hosepipe collecting the