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Significance of Using Uniform Hazard Spectra on Seismic Response Behavior of Bridges

by
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B.Eng. (Civil Engineering)
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A thesis submitted to

The Faculty of Graduate Studies and Research
in partial fulfillment of the requirements

for the degree of
Master of Engineering

**Department of Civil and Environmental Engineering
Carleton University, Ottawa
November 2000**

The master of Engineering Program in Civil and Environmental
Engineering is a joint Program with the University of Ottawa,
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Significance of Using Uniform Hazard Spectra on
Seismic Response Behavior of Bridges
submitted by
Sara Ahrari, B.Eng.

in partial fulfillment of the requirements for the
degree of Master of Engineering

Chair, Department of Civil and
Environmental Engineering

Thesis Supervisor

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Carleton University
Ottawa, Ontario, Canada
November 2000

Abstract

The thesis studies the significance of using uniform hazard spectra (UHS) in the design of bridges, and in particular the soil amplification effects and the ductility demand of bridges are investigated. The UHS compatible ground motions with 10% and 2% probability of exceedance in 50 years for Montreal in the east and Vancouver in the west of Canada are used as input motions in the analysis of the responses of bridge. The results obtained from using the uniform hazard spectra are compared to the results from actual recorded earthquakes. The results of study on the soil amplification effects are correlated with the seismic code requirements of five major bridge design codes. The nonlinear behavior and ductility demands of two bridges are evaluated in the present study. The effects of using the uniform hazard spectra compatible ground motions with different probability levels of occurrence on the displacement ductility demand of the bridges are investigated.

Dedicated to

The students of Baha'i Institute for Higher Education in Iran

&

*The youth all around the world
who are deprived of proper education.*

Acknowledgments

I would like to express my sincere gratitude to my supervisor Professor D.T. Lau for his guidance, advice and interest during the course of this research. Special thanks are also owed to my co-supervisor Professor G. Hartley for his assistance, his encouragement and advice. I also offer many thanks to Professor G. Atkinson for providing the simulated ground motions and suggestions. I would like to extend my sincere thanks to Professor K.T. Law for his valuable comments of the research. Special thanks also go to the staff in the Civil and Environmental Engineering Department Office at Carleton University for their continuous assistance and cooperation.

I would particularly wish to thank and express my deepest appreciation to my parents, my brother, Siamak and his wife, Checkameh. Without their loving support and continuous encouragement, it would not possible for me to pursue this work.

My most grateful acknowledgment is to the National Spiritual Assembly of Canada for being a wonderful source of love and inspiration and for providing the financial support throughout this project.

Many thanks to my friends and family from Albania, Australia, Bulgaria, Canada, China, Egypt, Finland, France, Germany, Greenland, India, Iran, Mauritius Island, Pakistan, Romania, Sweden, and Vietnam, whose love, prayers, patience and support have been beyond and above imagination. Thank you also for providing jokes, e-mails, mails, daily phone calls, discussions and other activities which all made my study at Carleton University and my stay in Canada a memorable experience.

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Chapter 1

Introduction

1.1 General

The dynamic behavior and performance of highway bridges under seismic actions is a subject of main concern to bridge engineers designing structures for seismically active areas. As many of these highway bridges are part of transportation networks in different countries, damage to these bridges may not only cause the loss of human life but also cause severe economical loss from the damage itself and from loss of productivity. Severe seismic damage or collapse of highway bridges may also slow down or stop immediate post-disaster rescue efforts.

The procedures and practices of earthquake resistant design of bridges are continually updated and improved upon to reflect the recent advances in earthquake engineering and new technology developed in the design of bridge systems. In particular, some part of the code provisions are based on experiences and observations on the performances of highway bridges during past earthquakes. Consequently, the lessons learned from past seismic activities are the basis for the improvement of these parts of the seismic codes for bridges.

In development of the earthquake resistant design concepts and codes for highway bridges, the 1971 San Fernando earthquake in southern California stands out as a major event. The 1971 San Fernando earthquake caused substantial damage to and

collapse of numerous modern and apparently well-designed highway bridges in addition to other damage. Following that earthquake, there was an upsurge in research interest into the seismic performance and design of bridge structures in different countries.

In the development of the earthquake resistant design concept and methodology, much of the research has focused on the behavior and performances of buildings. Although there are many similarities in the seismic design philosophies of buildings and bridges, because of basically different lateral load resisting systems in these structures, differences in design requirements and limitations can be significant. Firstly, bridges are relatively light structures and thus dynamic behavior of bridges is not significantly influenced by soil-structure interaction effects (Ciampoli et al. 1997) as compared to buildings. Secondly, the consequence of structure failure in bridge can be quite different from that in a building structure. Because of lack of redundancy in a typical highway bridge system, the failure of one structural member or connection between elements in a bridge usually has more serious consequences and may result in the overall collapse of the structure, which is unlikely to happen in a building where usually the degree of redundancy is high.

The effect of local soil conditions on the characteristics of input ground motions is one of the major concerns in the earthquake resistant design of bridges, particularly for bridges located in or near areas of river sediment or soft soil. The failure of many highway bridges in the past has occurred because of the amplification effect of underlying soil on bedrock during seismic ground motions. Experiences and observations from major earthquakes in California and other parts of the world have shown that bridges built on clay are especially susceptible to failure during earthquakes. The 1985 Mexico City earthquake and 1989 Loma Prieta earthquake are examples of seismic actions that caused damage because of the soil amplification effects inherent in

the soft soils. Severe damage and collapse of bridges were also observed due to local soil amplification effects during the 1994 Northridge earthquake where bridges were built on stiff or dense granular materials of 10m to 17m deep.

Analytical studies of the local soil effects have concluded that soil parameters and geologic conditions that may have significant effects on the characteristics of the ground motions at a site include:

- type and thickness of soil layers on top of the bedrock;
- variation of soil characteristics and properties (shear modulus, mass density and material damping) along the depth;
- nonlinear variation of soil properties under strong earthquakes; and
- lateral irregularity and surface topography at the site.

In earthquake resistant design, the soil effect on ground motions is generally represented approximately by a so-called "soil factor". Except for the degree of detail in different seismic design codes, this factor depends on the characteristics and depth of the soil deposit as well as the frequency characteristics of the bedrock motions. Extensive research has been carried out by Seed et al. (1976), Heidebrecht and Lu (1987), Naumoski et al. (1988), Elhamdi et al. (1990), Heidebrecht et al. (1990), Rahgozar (1993), Tinawi et al. (1993), Kokusho et al. (1996) and others to attain a better understanding of soil effects on ground motion and to introduce more reliable soil factors to be used in seismic design provisions of buildings and bridges.

Another important factor, which is recognized to have an important role in seismic behavior and earthquake resistant design of bridges, is structural ductility. In the inelastic ductile design concept, the design strength of the structure is reduced, considering that the structure will behave in a ductile manner and has sufficient redundancy for the redistribution of seismic loads after initial yielding failure of the

structure. The current practice in earthquake resistant design of highway bridges relies on the capability of the ductile components in a bridge system, such as column or piers, to absorb or dissipate the seismic energy imparted by the strong ground motions. To properly implement the ductile design concept, it is important to have a clear understanding and knowledge of the dynamic response and behavior of the designed bridge systems. A bridge structure should be designed and detailed in such a way that the ductile components, such as bridge piers and columns, during a severe earthquake will have sufficient reserved ductility capacity to accommodate the expected ductility demand of system.

The current national design code for highway bridges in Canada is the CSA Standards CAN/CSA-S6-88 (Canadian Standards Association 1988). However, this code does not address seismic issues adequately and therefore bridge engineers in different provinces of the country tend to use provisions from other bridge seismic codes i.e. the AASHTO (American Association of the State Highway and Transportation Officials 1996) and the OHBDC (Ontario Highway Bridge Design Code 1991) (Naumoski 1999). A new design code, the Canadian Highway Bridge Design Code (CHBDC, Canadian Standard Association 1999) is currently being developed to be the first national bridge design code in Canada, which includes extensive revisions to the seismic design provisions. Many investigations have been carried out or are currently underway to develop seismic provisions that will provide a more consistent level of safety against seismic hazards across Canada. A new approach has been recently proposed (Adams et al 1999) to quantify the seismic hazard by uniform hazard spectra (UHS) in Canada. This new approach incorporates a significant increment of earthquake data, recent research on source zones and earthquake occurrence, together with complementary research on strong motion relations. Therefore, it is of great interest to investigate the effects and verify the consequences of adopting the UHS approach in the seismic provisions of bridge design codes in Canada.

1.2 Objective and Scope of the Thesis

The objective of this study is to investigate the effects and possible methods of incorporating the results of the researches and technologies in the different aspects of seismic design of bridges. In particular, the significance of using uniform hazard spectra (UHS) on soil amplification effects and the ductility demand of bridges are investigated. The median values of UHS with 10% and 2% probability of exceedance in 50 years for Montreal in the east and Vancouver in the west of Canada are considered. The results obtained from using the uniform hazard spectra are compared to the results from the actual recorded earthquakes. The results of soil amplification effects are used to verify the adequacy of the different approaches and factors, which represent the soil amplification effects in the seismic provisions of five major bridge design codes. The nonlinear behavior and ductility demands of two bridges are also evaluated in the present study. The effects of using input ground motions, with different probability levels of occurrence, on the displacement ductility demands of bridges considered are investigated. The present study also includes the effects of soil amplification on the displacement ductility demand of bridges. It is expected that the results of this study provide a better knowledge of the performance of bridges under seismic actions; which can be included in the seismic provisions of the bridge design codes.

The thesis is divided into six chapters.

1. Introduction: A brief introduction on the motivation, objective and scope of the present study are provided in this Chapter.
2. Seismic Design of Bridges: The design philosophies, the approaches adopted to determine the seismic design loads and the design parameters of the seismic provisions in five major bridge design codes are reviewed in this Chapter.

3. Soil Amplification Effects: In this Chapter the soil amplification effects of four different types of soil subjected to various ground motions are investigated. The parameters considered in the soil amplification effects are: soil type; soil depth and the frequency content of the ground motions.
4. Response Spectra of Design Code Provisions: The design response spectra adopted in the bridge design codes considered in the present study are compared and their adequacies are evaluated in this Chapter.
5. Ductility Demand of Bridge Systems: The nonlinear behavior of two bridges are investigated in this chapter. The ductility demands of the components that are expected to behave inelastically are estimated using different ground motions.
6. Conclusion: The summary and conclusions are presented in this Chapter.

Chapter 2

Seismic Design of Bridges

2.1 Introduction

The design procedures and practices for earthquake resistant design of bridges are continuously improved and updated to reflect the latest advances in earthquake engineering and new technologies developed in the design of bridge systems. In particular, some of the seismic design provisions in the design codes of bridges, as in building codes, are based on experiences and observations on the performance of structures during earthquakes. Consequently, the data collected and lessons learned from the performance of structures in major earthquakes since the last revisions or updates of the design codes provide valuable new information and form basis for the improvement of these parts of the seismic design codes of bridges.

The development of seismic bridge design codes in North America can be divided into two periods, the period prior to the 1971 San Fernando earthquake in southern California and after. The seismic design code published by the American Association of State Highway Officials (AASHTO 1965) was a typical bridge design code used by bridge engineers prior to the 1971 San Fernando earthquake. In this code, the design code procedures for highway bridges were the same as those for building design at the time. The seismic action was represented by an equivalent lateral static force applied at the center of gravity of the structure. The equivalent lateral static load was taken between 2% to 6% of the dead weight of the structure depending

on the type of foundation and the allowable loads transmitted to the foundation. The 1965 AASHTO code did not take into account the influences of the site geological conditions on the intensity and characteristics of the ground motions and thus their effects on the dynamic responses and expected ductile behavior of the bridges. The 1971 San Fernando earthquake caused the collapses of five major bridges, and the severe damage to 37 modern concrete highway bridges. Following that earthquake there was significant increase in research interest to look into the seismic performance and design practices of bridge structures both in the US and abroad. Immediately after the 1971 San Fernando earthquake, the California Department of Transportation (CALTRAN) initiated many investigative and research efforts with the aim to improve the seismic design methodology for highway bridges (Gate 1976, CALTRAN 1977). As a result of these concerted efforts, significant improvements to the earthquake resistant design of highway bridges have been achieved. These achievements were later incorporated into the seismic design codes and standards by the Applied Technology Council (ATC 1981) and the 1983 AASHTO design code by the American Association of State Highway and Transportation Officials (AASHTO 1983).

Extensive research on earthquake engineering of bridges has been also carried out in New Zealand in the last three decades leading to better understanding of the behavior and performance of bridges during earthquakes. Significant advances in the understanding of the response behavior of bridges during strong ground shakings include: behavior characteristics and the enhancements to the compressive strength and the ability to sustain large deformation of confined concrete; the flexural strength and ductility characteristics of bridge columns; the strength and behavior of prestressed and steel-encased piles under seismic loading; and the behavior of concrete columns under shear loads. The research results have been incorporated into the Bridge Manual of Transit New Zealand (TNZ code) published by Transit New Zealand [68] which reflects the state-of-the-art practices in seismic design of bridges in that country.

In Canada, a significant step in the development of the code provisions for seismic design of bridges was the publication of the first edition of the Ontario Highway Bridge Design Code in 1979 (OHBDC 1979). The Ontario Highway Bridge Design Code was specifically developed for the design requirements of bridges in Ontario. The Ontario Highway Bridge Design Code was specifically developed for the design requirement of bridges in Ontario. Another highway bridge design code Canadian Standards Association-56 (CSA-56) published by the Canadian Standards Association (CSA 1988) contains provisions which are applicable to bridge design across Canada. The seismic design procedures in the CSA-56 code are adapted from the seismic design code provisions of the 1985 National Building Code of Canada (NBCC 1985). It is recognized from the observed damage to and observations of the performance of bridges in recent major earthquakes that some important aspects of the earthquake responses of bridges are not addressed adequately by the CSA-56 code (Taylor 1999). A new design code, the Canadian Highway Bridge Design Code (CHBDC), is currently being developed to be the first national bridge design code in Canada, which includes extensive revision to the seismic design provisions. The basic framework of the seismic design procedures stipulated in the CHBDC code are similar to those in the 1994 American Association of State Highway and Transportation Officials Load and Resistance and Factor Design [3].

In view of the many advances in the seismic analysis and design of bridges in recent years and the development of the new national bridge code in Canada, it is therefore important to evaluate the impacts of these advances, and new information and bridge performance data on the bridge design practices in Canada. In the present study, the seismic requirements in some of the existing bridge design codes commonly used for the design of bridges in high seismic areas are discussed. In the following sections, the seismic design provisions of the codes selected for comparison are briefly

reviewed.

2.2 Review of Seismic Design Code for Bridges

The design philosophies, the approaches adopted to determine the seismic design loads and the design parameters (which quantify the demands from the assumed seismic actions specified in the codes) of the seismic design code provisions in five major bridge design codes are presented in the following sections. The five bridge design codes are selected because they represent the generally accepted practices in ductile earthquake resistant design of bridges, and because of the ease of comparisons of these requirements among the group. The selected codes in the study are:

- Standard Specification for Highway Bridges by the American Association of State Highway and Transport Officials Load and Resistance Factor Design Code (AASHTO, 1997)
- Proposed Canadian Highway Bridge Design Code (CHBDC, 1998)
- Bridge Design Specification, California Department of Transportation (CALTRAN, 1999)
- Ontario Highway Bridge Design Code (OHBDC, 1991)
- Bridge design Manual of Transit New Zealand (TNZ, 1995)

A summary of the load determination approaches of the different design codes is presented in Table 2.1.

2.2.1 AASHTO Design Code (USA, 1997)

Design Philosophy

In accordance to the design procedures of the 1997 AASHTO design code, the expected elastic forces in the bridge members and connections generated under the

maximum probable earthquake are determined from structural analyses of the design bridge model subjected to the design seismic actions. Different appropriate reduction factors are used to obtain the design forces for the structural members or connections. The reduction factors used are different for different types of members and connections based on the expected strength and ductility capacities of the individual types of design component. Thus, as a result of this capacity design approach different types of structural components in the superstructure may achieve different levels of performance during earthquakes as a consequence of using different “design seismic coefficients” in their design. The concept of capacity design is also employed in the design of the bridge foundation. The bridge foundation is designed to withstand the smaller of:

1. the maximum elastic forces determined by considering the bridge system, both the superstructure and the substructure, behaving elastically during the earthquake; and
2. the forces resulting from the formation of plastic hinges in the columns of the superstructure.

To avoid the loss-of-span failure of bridge girders, which has been observed in many cases of bridge failure in past earthquakes, the AASHTO code requires a minimum support seat length for the girders in their supports by the abutment or bridge piers. This code also requires, where appropriate, vertical hold-down devices and positive linkage elements for the effective transfer of loads from the superstructure members to the substructure.

Design Force Level and Reduction Factors

According to the AASHTO seismic provisions, the earthquake loads are determined as the product of the equivalent weight of the superstructure and the elastic seismic response coefficient, C_{sm} , adjusted by an appropriate response modification

factor. The seismic response coefficient, which is discussed in more detail later in this section, is determined dependent on a number of parameters, which includes: the acceleration coefficient, A ; importance category, IC ; seismic performance zone, Z ; site effect, S ; and response modification factor R .

Acceleration Coefficient, A : The seismic risk at a site is represented, in the AASHTO code by an acceleration coefficient, A . The value of the acceleration coefficient, A , across the US is presented in a contour map. Linear interpolation is permitted in the use of the contour map to determine the seismic risk of sites located between the contour lines.

Importance Categories, IC : All bridges designed using the AASHTO code are assigned to an importance classification as follows

- essential bridges
- critical bridges, or
- other bridges

The first category of essential bridges are those bridges that should, as a minimum, remain open to emergency traffic immediately after the design earthquake, which is taken as a seismic event with a return period of 475 years. On the other hand, some bridges may be required to have a higher performance and may have to remain open to traffic immediately after the design earthquake, and be usable by emergency vehicles, after a rare strong earthquake, for example, an event with a 2500 year return period. These are classified as critical bridges. The classification of a bridge for the purpose of design is decided based on current functional requirements, as well as consideration of possible future changes in conditions and requirements governing its use.

Seismic Performances Zones, Z : Depending on the specified acceleration coefficient of a site (A) the bridge is assigned to one of the four seismic zones in accordance to the requirements presented in Table 2.2. These seismic zones reflect the variation in seismic risk which leads to the different requirements in methods of analysis, minimum support lengths, column design details and foundation and abutment design procedures.

Site Effect, S : The effect of site conditions on the basis of the elastic response spectrum are included through the site coefficient, S , as shown in Table 2.3.

Site effects on structural response are associated with the local soil conditions of the site. Four types of soil profile are provided in the definition of the parameter of site coefficient. These soil profiles are representative of different subsurface conditions, which are selected on the basis of a statistical study of spectral shapes developed with similar soil condition close to the seismic source zones in past earthquakes.

Response Modification Factor, R : For the design of the bridge components, the design force is obtained from the maximum seismic design force calculated using the appropriate method of analysis, and then divided by the response modification factor, R , presented in Tables 2.4 and 2.5. In the AASHTO design procedures, the response modification factor, R , depends on the bridge importance category. It is also noted that R is different for different components of the same bridge. This is in contrast to the generally accepted seismic design practice of buildings, wherein the building as a whole is assigned a single value of response reduction factor. The same response reduction factor is used in the determination of the seismic design forces of all the components in the building. Another important difference in the practice of seismic design of bridges as compared to the building design practices is that the response reduction factor, R , can be less than one, and be as low as 0.8 for the connections,

which means for critical components such as bridge connections, they are designed to resist loads higher than the corresponding maximum expected elastic design forces.

Elastic Seismic Response Coefficient

The maximum elastic seismic response coefficient in the AASHTO code is provided by using the standard shapes of the response spectrum for the specified soil profiles. Since an earthquake may excite several modes of vibration in a bridge, the effect of the higher modes are considered, in the AASHTO code by finding the elastic coefficient for each relevant mode using the corresponding period of vibration. This is different from the seismic provisions in most other bridge codes where the effect of higher modes is not considered in defining the seismic load. In the AASHTO code, the elastic seismic response coefficient, C_{sm} , is defined for the m^{th} mode of vibration from the spectra of the design earthquake for an equivalent single-degree-of-freedom system with 5% damping as follows

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A \quad (2.1)$$

In this equation T_m represents the period of vibration of the m^{th} vibration mode in seconds, A is the acceleration coefficient and S is the site coefficient. The determination of the period of vibration, T_m , is based on the nominal, unfactored mass of the component of the structure.

There are some exceptions in applying Eq. 2.1 specified in the code. It is stated that for bridges located on soil profile III or IV and in areas where the coefficient “A” is not less than 0.30, C_{sm} need not exceed 2.0A.

For soil profiles III and IV, and for modes other than the fundamental mode which have periods less than 0.3 second, C_{sm} is taken as

$$C_{sm} = A(0.8 + 4.0T_m) \quad (2.2)$$

If the period of vibration for any mode exceeds 4.0 second, the value of C_{sm} for that mode is taken as

$$C_{sm} = \frac{3AS}{T_m^{4/3}} \quad (2.3)$$

Methods of Analysis

The AASHTO code allows four different analysis procedures to be used in determining the minimum requirements on seismic design forces or seismic action demands. These are in order of increased complexity and level of accuracy the “uniform load method” (UL), the “single mode elastic spectral method” (SM), the “multi-mode elastic spectral method” (MM) and the “time history method” (TH). The requirement on the selection of analysis method is based on the consideration of the seismic zone where the bridge is located, the number of spans, the importance category and the complexity of the bridge, as defined in the Table 2.6.

Combination of Orthogonal Seismic Actions

Seismic actions are three dimensional in nature. A simple way to account for the multi-directional effect of the horizontal seismic actions, the elastic seismic forces and moments obtained from analyses is to consider the seismic action separately in two perpendicular directions and combine them by taking 30% of the seismic demand resulting from applying the seismic action in one direction to the demand resulted from the other, and vice versa. The bridge components are designed against the worst requirement. A combination including the vertical component is not required according to the AASHTO LRFD (1997) code.

Load Combinations

The design approach in AASHTO LRFD (1997) employs load factors and load combination factors in procedures similar to the limit states design procedures. In

the limit states design procedure the total factored loads are taken as:

$$Q = \eta \sum \gamma_i q_i \quad (2.4)$$

where η = load modifier (between 0.95 to 1.16),

q_i = specified loads, and

γ_i = specified load factor.

The case of load combination which includes earthquake, regarded as Extreme Event I in AASHTO LRFD code, is expressed as follows

$$q = \gamma_p(DC + DD + DW + EH + EV + ES) + \gamma_{EQ}(LL + IM + CE + BR + PL + LS) + WA + FR + EQ \quad (2.5)$$

where γ_p , DC, DD, DW, EH, EV, and ES are permanent loads, as specified in Table 2.7. where:

γ_{EQ} = earthquake load factor to be determined on a project specific basis (recommended value is 0.5);

LL = vehicular live load ;

IM = vehicular dynamic load allowance ;

CE = vehicular centrifugal force ;

BR = vehicular braking force ;

PL = pedestrian live load ;

LS = live load surcharge ;

WA = water load and stream pressure ;

FR=friction ;

EQ=earthquake.

2.2.2 Canadian Highway Bridge Design Code (Canada, 1998)

The design philosophy, the seismic design loads and the corresponding factors in the proposed Canadian Highway Bridge Design Code (CHBDC, 1998) are adapted from the American Association of State Highway and Transport Officials Load and Resistance Factor Design, 1997 (Mitchell et al. 1998). However, there are some differences in the determination of the seismic parameters, which are explained in the following sections.

Design Force Level and Reduction Factors

Zonal Acceleration Ratio, A : In the proposed CHBDC code, the zonal acceleration ratio, A , which prescribes the level of seismic hazard, is obtained from tabulated data in the standard for the bridge site, or by using the peak horizontal acceleration, PHA, determined from the seismic map of Canada. For sites close to active faults it is required that a qualified specialist be consulted to determine the appropriate zonal acceleration ratio for the design.

Importance Category, I : The importance categories considered in the CHBDC code are

- lifeline Bridges
- emergency-route Bridges, or
- other Bridges

Lifeline Bridges are those that carry or cross over routes that are supposed to remain open to all traffic after the design earthquake. The design earthquake is considered an event with 10% probability of exceedance in 50 years, equivalent to a 15% probability of exceedance in 75 years and a return period of 475 years. These bridges are also required to be usable by emergency vehicles immediately after a 1000-year return period event which corresponds to 7.5% probability of exceedance in 75 years.

Emergency-route Bridges are those that carry or cross over routes that need to have a higher performance level than typical bridges. They are required to remain open to emergency vehicles immediately after the design earthquake.

Seismic Performance Zones : Using the zonal acceleration ratio, A , the CHBDC code categorizes bridges to one of four seismic performance zones presented in Table 2.8.

Response Modification Factor : To account for the ductility in the structural elements, the seismic force resulting from elastic analysis is divided by the appropriate response modification factor, R , as specified in Table 2.9. The reduction factors in CHBDC code are not dependent on the importance category of the bridge. This is another difference between the seismic provisions in the CHBDC code and the specification in the AASHTO code.

Elastic Seismic Response Coefficient

The CHBDC seismic provisions define the elastic response coefficient used in the seismic design force calculation with a similar formula as that in the AASHTO LRFD design code. According to the CHBDC code, the elastic response coefficient,

C_{sm} , for the m^{th} mode of vibration is specified as

$$C_{sm} = \frac{1.2ASI}{T_m^{2/3}} \leq 2.5AI \quad (2.6)$$

The definitions of the parameters used in these formulas are the same as in the AASHTO design code. A difference with the AASHTO code is in the determination of the seismic demand, where the seismic response coefficient is modified by including the importance parameter of the bridge.

Methods of Analysis

According to the CHBDC design code, seismic analysis is not required for single-span bridges regardless of the assigned seismic performance zone, except for single-span truss bridges in seismic performance Zones 2, 3, and 4, which are required to be analyzed in a manner similar to that for regular, multi-span bridges.

For multi-span structures, four different analysis procedures, which are similar to the procedure from AASHTO LRFD code (1997), are considered acceptable as the minimum analysis requirement based on the importance category and complexity of the bridge, as presented in Table 2.10. These procedures are “uniformed load method” (UL), “single mode elastic spectral method” (SM), “multi-mode elastic spectral method” (MM) and “time history method”(TH).

Bridges may also be analyzed using inelastic analysis such as static push-over analysis or time-history analysis. A regular bridge is defined as a bridge with less than seven spans, no abrupt or unusual changes in weight, stiffness or geometry and no large changes in these parameters from span-to-span or support-to-support (abutment excluded). All other bridges are considered as irregular.

2.2.3 CALTRANS Design Code (USA, 1999)

The California Department of Transportation (CALTRANS) publishes a design manual for the purpose of seismic design of freeway/highway bridges in the state of California, US. The CALTRAN design code is used as a model code for many other codes in the seismic design of bridges.

Design Philosophy

In the CALTRAN standard, the bridge structure is designed to resist earthquake motions by taking into consideration the site location from the active faults, the seismic response of soil at the site, and the dynamic response characteristics of the design bridge.

The requirements presented in the following sections are considered as minimum requirements for the design. The designer is required to determine the appropriate methods and levels of refinement necessary to design and analyze each bridge on a case-by-case basis.

Demands on Structure Components

Ground Motion Representation : According to the CALTRAN seismic provisions, the elastic response spectra of a site for the maximum credible event(s) are determined by the product of peak rock acceleration, A , the acceleration spectra in rock, R , and the soil amplification factor, S .

Spectral Acceleration: The procedure to derive the seismic design load in the CALTRAN code is based on the deterministic ARS approach. The determination of the ARS parameters are described in the following sections.

Peak Rock Acceleration, A: The deterministic peak acceleration on rock, A , is specified in the CALTRAN Seismic Hazard Map.

Rock Spectra, R: The rock spectra are magnitude and distance dependant. Acceleration spectra ranging in values between 0.1 and 0.7g (in 0.1g increments) for three magnitude groups of earthquakes ($M6.5 \pm 0.25$, $M7.25 \pm 0.25$ and $M8.0 \pm 0.25$) are specified in the code. These spectra are for California-type rock and correspond to NEHRP Soil Profile Type B. These curves are a reasonable upper bound of the spectral values obtained using various spectral relationships.

Site Modification Factor, S: Table 2.11 summarizes the soil profile types considered in this code, which are the same as those adopted in the 1994 NEHRP Provisions.

Horizontal Ground Motions: In this standard earthquake effects are determined from horizontal ground motion applied by one of the following methods.

Method 1: The ground motion is applied in two orthogonal directions along a set of global axes, and then combined by taking 30% of the seismic demand resulted from applying the seismic action in one direction to the demand resulted from the other, and vice versa.

Method 2: The ground motion is applied along the principal axes of individual structural components. Here, the ground motion must be applied at a sufficient number of angles to capture the maximum deformation of all critical components.

Vertical Ground Motions: This code does not require a vertical acceleration response spectral analysis for ordinary standard bridges. For bridge sites where the

peak rock acceleration exceeds 0.5g, it is required that an equivalent static vertical load be applied to the superstructure to estimate the effect of vertical acceleration. A case-by-case determination on the effect of vertical load is required for non-standard and important bridges.

Vertical/Horizontal Load Combination: A combined vertical/horizontal load combination is not required for the seismic design of standard bridges.

Damping: An elastic ARS curve for an equivalent single-degree-of-freedom system with 5% damping is used to determine the acceleration for ordinary standard concrete bridges. A damping ratio of 10% may be assumed in the design of bridges that are heavily influenced by energy dissipation at the abutments and are expected to respond in a manner similar to the behavior of single-degree-of-freedom systems. A reduction factor, R_D is applied to the 5% damped ARS coefficient to determine the displacement demand. The displacement demand, R_D , when a damping ratio other than 5% is used, is obtained using the following formula:

$$R_D = \frac{1.5}{40C + 1} + 0.5 \quad (2.7)$$

where C the is damping ratio, having a maximum amount of 10%.

Target Displacement Ductility Demand

Ductility is defined as a local measure of the post-yield or inelastic capacity of a structural member. Since it is difficult to distinguish the local member displacement from the system displacement in the current available analytical tools, the displacement ductility demand in this code is specified as a performance measure that captures how much inelastic response can be expected from an individual member based on the global displacement of a frame. The displacement ductility is then defined mathematically by the equation as follows

$$\mu_D = \frac{\Delta_D}{\Delta_Y} \quad (2.8)$$

where:

Δ_D = the estimated global frame displacement demand; and

Δ_Y = the yield displacement of the subsystem from its initial position to the formation of plastic hinge.

The displacement ductility demand of any particular component is limited to the target values presented in Table 2.12.

Force Demand

The CALTRAN seismic provisions require the design structures to resist the internal forces generated when the structure reaches its collapse limit state. The collapse limit state is defined as the condition when a sufficient number of plastic hinges are formed in the structure to form a local or global collapse mechanism.

Methods of Analysis

To assess the force and deformation demands and capacities on the structural system and its individual components, different seismic analysis procedures can be followed.

For ordinary standard bridges the following methods are employed to estimate the displacement demands:

- equivalent static analysis (ESA);
- elastic dynamic analysis (EDA); and
- inelastic static analysis (ISA)

It is stated in the CALTRAN code that *ESA* can be used when a more sophisticated dynamic analysis will not provide additional insight into the behavior. It is especially suited for bridges or individual frames with more or less uniform spans and uniformly distributed stiffness where the response can be captured by a predominant

translational mode of vibration. The seismic load is assumed as an equivalent static horizontal force applied to the individual frames. The total applied force is considered to be equal to the product of the ARS and the tributary weight. The horizontal force is applied at the vertical center of mass of the superstructure and distributed horizontally in proportion to the mass distribution.

EDA is used where *ESA* does not provide an adequate level of sophistication to estimate the dynamic behavior. A linear elastic multi-modal spectral analysis utilizing the appropriate response spectrum may be employed. The number of degrees-of-freedom and the number of modes considered in the analysis are established so as to capture at least 90% of the mass participating in the dynamic responses of the bridge in the longitudinal and transverse directions. A minimum of three elements per column and four elements per span are required in the linear model.

ISA, commonly referred to as “push over” analysis, is allowed to determine the reliable displacement capacities of a superstructure or frame as it reaches its limit of structural stability.

Bridge systems with irregular geometry, in particular: curved and skewed bridges; bridges having multiple transverse expansion joints or massive substructure components; and bridges on foundations of soft soil can exhibit dynamic response characteristics that are not necessarily obvious and may not be captured in a separate system analysis. The code requires structural system or global analysis to be used in these cases in order to capture the response of the entire bridge. The effects of soil/foundation flexibility are required to be included, if they are significant.

Load Combination

The design approach in CALTRANS (1999) employs combinations of different group loads. The earthquake design load is combined with other design loads according to the following equation:

$$GroupLoad = 1.0(D + B + SF + \alpha_E E + PS + EQ) \quad (2.9)$$

where: D is the dead load; B is the load effect due to buoyancy; SF is stream flow pressure; α_E is the load factor for loads due to earth pressure which may vary from 0.5 to 1.5; E is the load due to earth pressure; PS is the load due to prestressing effect, and EQ is the earthquake load.

2.2.4 OHBDC Design Code (Canada, 1991)

Design Philosophy

The publication of Ontario Highway Bridge Design Code (OHBDC) in 1979 was a significant step in the development of the code provisions for seismic design of bridges in Canada. The current OHBDC (1991) design code takes into account the seismic action by modifying a response coefficient, C , to permit the use of the regional seismicity data of Ontario with a comparable level of safety in areas across Canada. The OHBDC does not consider the effect of the multiple components of the seismic actions.

Design Coefficients

Frame Coefficient, F : The effect of the structure is taken into account by considering a frame coefficient, F in the OHBDC code. F is equal to 1.0 for structures where single columns or piers resist the horizontal loads and 0.8 for structures where continuous frames resist horizontal loads applied along the frame.

Importance Factor, I : The OHBDC code, considers two importance factors for different bridges. It assumes I to have a value of 1.3 for all bridges designed for post-disaster service and a value of 1.0 for all other structures.

Response Coefficient Factor, C : The effects of seismicity of the site and the underlying soil as well as flexibility of the bridge are taken into account by the response coefficient factor C . The major difference between OHBDC and other codes is that the coefficient C is calculated based on the velocity-related zonal ratio v instead of the maximum expected zonal acceleration a . The OHBDC makes reference to the seismic zoning maps in the National Building Code of Canada. (1985). This zoning maps are developed on the basis of two ground motion parameters that characterize the intensity of ground motion, peak acceleration a and peak horizontal velocity v . The acceleration and velocity maps provide independent ground motion reference levels to better reflect the seismic risk from large earthquakes at a large distance and moderate earthquakes nearby with a probability of exceedence of 10% in 50 years. The zonal velocity ratio varies from 0.05 to 0.3 m/s. The design spectra are developed for four categories of alluvium soil depths:

1. thickness of alluvium 0-3 m;
2. thickness of alluvium 3-25 m;
3. thickness of alluvium 25-45 m;and
4. thickness of alluvium > 45 m.

The effect of structural ductility and structural damping are considered in the response coefficient curves of C .

Methods of Analysis

The OHBDC code does not require any detailed seismic analysis for single span bridges and bridges where the zonal velocity ratio v , is less than 0.08. For multi-span bridges located at sites where the seismic zonal velocity v , is equal to or greater than 0.08, a detailed seismic analysis is required to be performed using one of the following methods:

- equivalent static method
- response spectrum method

The relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure are required to be included in this analysis.

Load Combination

The group load combination including earthquake in OHBDC code is expressed as follows:

$$GroupLoad = \alpha_D D + \alpha_E E + \alpha_P PS + 0.7W + 1.3EQ \quad (2.10)$$

where: α_D and α_P are load factors for dead loads and prestressing effects. The variation of the load factor α_D can range from 0.65 to 1.5 depending on different dead load types, and 0.5 to 1.25 for α_E depending on the characteristic of the earth pressure load, and 0.95 to 1.05 for α_P . W is the wind load.

2.2.5 TNZ Bridge Design Code (New Zealand, 1995)

The TNZ bridge design specification is published by Transit New Zealand for the purpose of seismic design of highway bridges in New Zealand. The Bridge Manual of Transit New Zealand, here in after called the TNZ code, today stands as the design code which reflects the state-of-the-art practice in seismic design of bridges.

Design Philosophy

The TNZ code is based on the concept of capacity design. The design approach involves the selection of an intended mode of structural or failure pattern during strong shaking. The design and detailing of the members are then required which assumes that the structure behaves as intended. Sufficient strength capacity is provided elsewhere in the structure to ensure that the intended energy dissipation mechanism does indeed develop in the event of a major earthquake. The TNZ code takes into account the effect of ground motion in the orthogonal directions in the design of the bridge. The seismic performance requirements of the Transit New Zealand Code are as follows.

1. After the design return period event, the bridge is required to be usable by emergency traffic, although damage may have occurred, and some temporary repairs may be required. Permanent repair to reinstate the design capacities for both vehicle and seismic loading should be feasible.
2. After an event with a return period significantly less than the design value, damage is supposed to be minor, and there should be no disruption to traffic.
3. After an event with a return period significantly greater than the design value, the collapse of the bridge should not occur, although damage may be extensive. It is required that the bridge be usable by emergency traffic after temporary repairs and should be capable of permanent repair, although a lower allowable traffic load limit may be imposed after the repair.

Structural Action

The TNZ code categorizes each structure according to its structural action under the horizontal seismic loading into eight categories. These categories are defined with reference to relationship between the total applied horizontal loading and the resulting displacement of the center of mass of the whole superstructure, as follows:

1. ductile structure;
2. partially ductile structure (Type I and II);
3. structure of limited ductility demand;
4. structure of limited ductility capacity;
5. elastic structure;
6. structure incorporating mechanical energy dissipating devices;
7. structure "locked in" to the ground; and
8. structure on rocking piers.

Design Earthquake Loading and Ductility Demand

Site Subsoil Categories and Structural Performance Factor, S_p : The TNZ code defines three subsoil categories based on the low amplitude natural period, unconfined compressive strength of the soil and depth of the soil, as follows:

1. rock or very stiff soil sites;
2. intermediate soil sites; and
3. flexible or deep soil sites.

Each of these subsoil categories are assigned a structural performance factor, S_p . The values for S_p are 0.90, 0.80 and 0.67 respectively.

Importance Categories and Risk Factor, R : The TNZ code defines three importance categories for bridges depending on the average number of vehicles per day expected at the time of design, on the location (crossing or carrying motorways or railways), and on whether the design bridge is a part of the national or provincial highway. Based on the importance category, a risk factor, R is defined for determining

the design force. This factor is taken as 1.0, 1.15 and 1.3, in order of the importance of the bridge.

Zone factor, Z : A zone factor Z , which reflects the peak ground acceleration at the site, is given in the form of a contour map for the New Zealand island. The TNZ code specifies 0.6 for the a minimum value of Z and 1.2 for the maximum value.

Ductility, Inelastic Design Spectra

The TNZ code considers the different performance in ductility of different types of structures. It requires designers to use the actual characteristics of the structure to evaluate appropriate displacement ductility factor, μ , in the design. Indirect reference is made to the use of nonlinear analysis in obtaining the appropriate μ .

In the TNZ code provisions, a basic acceleration coefficient C_μ , is employed to reflect the response amplification due to the structure flexibility and a response reduction factor is used to account for the ductility and overstrength in the structural system. The coefficient C_μ is specified with reference to the level of displacement ductility assumed to be achieved by the bridge during strong earthquake. TNZ is one of the few codes to allow the use of inelastic spectra in design.

2.2.6 Methods of Analysis

The seismic analysis is performed by constructing the basic design spectrum given by the product of basic acceleration coefficient C_μ , zone factor Z and the risk factor R . It should be noted that no response reduction factor is being used in the process. The main component of response reduction factor, namely ductility reduction factor and overstrength, are accounted for through the use of the acceleration coefficient, C_μ . The TNZ code permits three types of analyses, namely:

- equivalent static force analysis;

- response spectrum analysis; and
- inelastic time history analysis.

The TNZ code specifically states that the design forces and moments in individual members, calculated by response spectrum analysis, should be at least 80% of the values obtained by the equivalent static analysis method.

Combination of Orthogonal Seismic Forces

A combination of the effects of orthogonal seismic actions is applied to the structural elements to account for the simultaneous occurrence of earthquake shaking in two perpendicular horizontal directions. The absolute values of effects (forces or moments) resulting from the analyses in one direction is combined with 30% of the absolute values of effects (forces or moments) in the orthogonal direction and vice versa to form two load cases. The design considers the worst requirement.

Load Combination

TNZ 1995 accounts for combinations of different group loads both for working stress and serviceability requirements and strength requirements. Load combination and allowable stresses for working stress and serviceability requirements, including earthquake, are as follows:

$$Group3A : DL + EP + OW + SG + ST + EQ + 0.33TP; \quad (2.11)$$

The percentage of basic stress allowed is 1.33. The load combination and load factor for strength requirements are:

$$Group3A : U = 1.00(kDL + 1.35(EP + OW) + SG + ST + EQ + 0.33TP) \quad (2.12)$$

in which:

k=1.3 or 0.8 whichever is more severe ;

DL= Dead Load, including superimposed dead loads;

EP= Earth pressure ;

EQ= Earthquake effects;

OW= Ordinary water pressure and buoyancy ;

SG= Shortening effect;

ST= Settlement;

TP= Temperature effects, overall and differential.

Table 2.1: Summary of Load Determination Approaches

Seismic Coefficients	AASHTO (1997)	CHBDC (1998)	CALTRAN (1999)	OHBDC (1991)	TNZ (1995)
Zoning	Z=1 to 4	Z=1 to 4	Included in the elastic response spectra(ARS) curves	Zoning map of Canada	Z=0.6 to 1.2
Acceleration coefficient, A	Seismic contour map of USA	Tabulated data for specific locations or seismic map of Canada	CALTRAN seismic hazard map	Seismic map of Canada	-
Importance factor, I	Essential, critical & other	Lifeline, emergency-route & other	NA.	I=1.0 or 1.3	I=1.0, 1.15 or 1.3
Soil amplification, S	Type 1 to 4 (S=1.0 to 2.0)	Type I to IV (S=1.0 to 2.0)	Type A to F	Included in the response coefficient curves, C	S=0.67 to 0.9
Response reduction factor	Piers or columns: R=1.5 to 5 Connections: R=0.8 or 1.0	R=1 to 5	Piers or columns: R=2 to 8 Connections: R = 0.8 or 1.0	Accounted for in the design response coefficient curves, C	Accounted for in ductility parameter in C_μ
Seismic Spectra	$C_{sm} = \frac{1.2AS}{T_m^{2/3}}$ where C_{sm} is elastic seismic response coefficient and T_m is the period of vibration of m^{th} mode	$C_{sm} = \frac{1.2AI/S}{T_m^{2/3}}$ where C_{sm} is elastic seismic response coefficient and T_m is the period of vibration of m^{th} mode	ARS curves (which are given for different soil types in three magnitude-distant group)	spectra of response coefficient, C , versus period (which is given based on zonal velocity for different soil types)	Spectra of basic acceleration coefficient C_μ according to fundamental natural period mode and structural ductility factor
Methods of analysis	1.Uniform load method 2.Single mode elastic spectral method 3.Multi-mode elastic spectral method 4.Time history method	1.Uniform load method 2.Single mode elastic spectral method 3.Multi-mode elastic spectral method 4.Time history method	3.Equivalent static analysis 2.Elastic Dynamic analysis 3.Inelastic analysis	1.Equivalent static load analysis 2.Response spectrum method 3.Inelastic time history analysis	1.Equivalent static load analysis 2.Modal analysis 3.Inelastic time history analysis
Combination of loads in dir. of the action	100% (in one dir.)+30%(in perpendicular dir.)	100% (in one dir.)+30%(in per. dir.)	1. 100% (in one dir.)+30%(in perpendicular dir.) 2.application of the load along the principle axes of the component	NA.	100% (in one dir.)+30%(in perpendicular dir.)
Seismic factor in group load combination	1.0	1.0	1.0	1.3	1.0

Table 2.2: Seismic Zones, AASHTO

Acceleration Coefficient	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.19 < A \leq 0.29$	3
$0.29 \geq A$	4

Table 2.3: Site Coefficient, AASHTO

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Table 2.4: Response Modification Factors - Substructure, AASHTO

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers - larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
i) vertical piles only	1.5	2.0	3.0
ii) with batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
i) vertical pile only	1.5	3.5	5.0
ii) with batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 2.5: Response Modification Factors - Connections, AASHTO

Connections	All Importance Categories
Superstructure to abutment	0.8
Expansion joints within a pan of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

Table 2.6: Minimum Analysis Requirements for Seismic Effects, AASHTO*

Seismic Zone	Single-Span Bridges	Multi-Span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

* *: no seismic analysis requires, UL: Uniformed load method, SM: Single mode elastic spectral method, MM: Multi-mode elastic spectral method and TH: Time history method.

Table 2.7: Load factors for permanent loads, γ_p , AASHTO

Type of load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.5	0.65
EH: Horizontal Earth Pressure		
• Active	1.5	0.90
• At-rest	1.35	0.90
EV: Vertical Earth Pressure		
• Overall Stability	1.35	N/A
• Retaining Structure	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid frames	1.35	0.90
• Flexible Buried	1.95	0.90
Structure other than Metal Culverts		
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

Table 2.8: Seismic Performance Zones, CHBDC

Range of Peak Horizontal Ground Acceleration (PHA), g, for 10% Probability of exceedance in 50 years	Zonal Acceleration Ratio, A	Seismic Performance Zone	
		Emergency-route and other bridges	Lifeline
$0.00 \leq \text{PHA} < 0.04$	0.00	1	2
$0.04 \leq \text{PHA} < 0.08$	0.05		
$0.08 \leq \text{PHA} < 0.11$	0.10	2	3
$0.11 \leq \text{PHA} < 0.16$	0.15		
$0.16 \leq \text{PHA} < 0.23$	0.20	3	3
$0.23 \leq \text{PHA} < 0.32$	0.30	4	4
0.32 or greater	0.40		

Table 2.9: Response Modification Factor, R, CHBDC

Ductile Substructure Element	R
Wall-type piers - in direction of larger dimension	2.0
Reinforced concrete pile bents	
• vertical piles only	3.0
• with batter piles	2.0
Single columns	
• ductile reinforced concrete	3.0
• ductile steel	3.0
Steel or composite steel and concrete pile bents	
• vertical piles only	5.0
• with batter piles	3.0
Multiple column bents	
• ductile reinforced concrete	5.0
• ductile steel columns or frames	5.0
Braced Frames	
• ductile steel braces	4.0
• normally ductile steel braces	2.5

Table 2.10: Minimum Analysis Requirements for Multi-Span Bridges, CHBDC*

Seismic Performance Zone	Lifeline Bridges		Emergency-route Bridges		Other Bridges	
	Regular	Irregular	Regular	Irregular	Regular	Irregular
1	N.A.	N.A.	None	None	None	None
2	MM	MM	UL	MM	UL	SM
3	MM	TH**	MM	MM	UL	MM
4	MM	TH**	MM	MM	SM	MM

* None: no seismic analysis requires, UL: Uniformed load method, SM: Single mode elastic spectral method, MM: Multi-mode elastic spectral method and TH: Time history method.

** Requires the permission on the Regulatory Authority. The use of multi-mode method seem to be appropriate for certain cases.

Table 2.11: Soil Profile Types, CALTRAN 1999

Soil Profile Type	Soil Profile <i>Description*</i>
A	Hard rock with measured shear wave velocity $\nu_s > 5000$ ft/s (1500 m/s)
B	Rock with shear wave velocity $2500 < \nu_s < 5000$ ft/s (760m/s< $\nu_s < 1500$ m/s)
C	Very dense soil and soft rock with shear wave velocity $1200 < \nu_s < 2500$ ft/s (360 m/s< ν_s 760 m/s) or with either standard penetration resistance $N > 50$ or undrained shear strength $s_u \geq 2000$ psf (100kPa)
D	Stiff soil with shear wave velocity $600 < \nu_s < 1200$ ft/s (180 m/s< $\nu_s < 360$ m/s) or with either standard penetration resistance $15 \leq N \leq 50$ or undrained shear strength $s_u \geq 2000$ psf (100 kPa)
E	A soil profile with shear wave velocity $\nu_s < 600$ ft/s (180m/s) or any profile with more than 10ft (3m) of soft clay, defined with plasticity index $PI > 20$, water content $w \geq 40$ percent, and undrained shear strength $s_u < 500$ psf (25 kPa)
F	Soil requiring site-specific evaluation: <ul style="list-style-type: none"> • Soil vulnerable to potential failure or collapse under seismic loading; i.e. liquefiable soils, quick and highly sensitive clays, collapsible weakly-cemented soils • Peat and/or highly organic clay layers more than 10 ft (3m) thick • Very high-plasticity clay ($PI > 75$) layers more than 25 ft (8 m) thick • Soft-to-medium clay layers more than 120 ft (36 m) thick

* The soil profile types shall be established proper geotechnical data.

Table 2.12: Target Displacement Ductility Demand according to CALTRAN

Component	Target Ductility Demand
Single Column Bents	$\mu_D \leq 4$
Multi Column Bents	$\mu_D \leq 5$
Pier Walls (weak direction)	$\mu_D \leq 5$
Pier Walls (strong direction)	$\mu_D \leq 1$
Pile Shaft (Type I)	$\mu_D \leq \text{Ductility of the column}$
Pile Shaft (Type II)	$\mu_D \leq 1$

Chapter 3

Soil Amplification Effects

3.1 Introduction

A soil deposit subjected to an earthquake has a tendency to focus the seismic energy of the bedrock motions within a narrow band of frequencies in the neighborhood of its natural frequency. As a result, the frequency content of the surface (free field) motions is amplified within that range, and thus poses a significantly higher seismic risk to structures with dominant vibration frequencies in the region than the risk associated with the corresponding bedrock motions. To better understand the seismic performance of structures during earthquakes, it is therefore important, from the structural engineering point of view, to study the resulting free-field surface motions from the bedrock input motions. This behavior of the soil deposit in modifying the seismic ground motions, typically referred to as the site amplification effect, is usually considered in the design provisions of the seismic design codes of structures. The first design code of structures considering the soil effect is 1932 Chilean code (Seed 1986). Previous studies by Seed et al (1976) and Mohraz (1976) have shown that the major factors that contribute to the site amplification effect are soil type, soil depth, and frequency content of the ground motion. It has been observed that the maximum level of acceleration of seismic ground motions recorded on soft soil sites can be significantly higher than those recorded on rock sites located at similar distances from the earthquake epicenter. The significance of the site amplification effect on the extent and severity of damage at soft soils has been observed in past

earthquakes, such as the earthquakes in Caracas 1967 (Seed et al. 1972) and Gediz, Turkey 1970 (Tezcan et al. 1977), Mexico 1985 (Boothe et al. 1986) and Turkey 1999.

The site amplification effect, however, is not limited only to soft soil deposit. Although liquefaction is a major concern of seismic hazards of sites with sand deposits, as demonstrated by the damage caused by the liquefaction of the foundation soils during the 1964 Niigata earthquake (Ohsaki 1966) and the 1964 Alaska earthquake (Seed 1968), research studies have shown that the site amplification effect can also be a significant contributing factor to the observed damage (Balendra and Heidebrecht 1986). Analysis of the 1971 San Fernando earthquake data shows that the location of the observed damage to residential buildings correlates well with the soil type of sand in less than 18m depth (Seed 1987). This seems to indicate that contrary to the belief that at a sand site the ground surface vibrates in unison with the underlying rock or has a free field motion only slightly amplified from the rock motion, significant site amplification effect occurs at those sites (Henderson et al. 1990). The phenomenon of the site amplification effect of sand deposits is similar to that of soft soil deposits. However, there are some significant differences in the behavior of the site effect of sand deposits compared to soft soil. Recent studies have shown that the level of shaking from the bedrock motions can be reduced in amplitude at liquefied sites of sandy deposits due to distraction of the seismic energy to different parts of the energy spectrum of the seismic motion and the isolation of the surface soil layer by the liquefied soil.

Many studies have been carried out to investigate the site amplification effect. Most of these studies focus on the behavior of buildings. It is well recognized that bridges behave differently than buildings in important aspects of mass and stiffness distributions, damping characteristics, and potential failure modes and damage patterns during earthquakes. The seismic design procedure developed for buildings may

not be appropriate for the design of bridges. Recognizing the differences in their behavior, the design of bridges are therefore governed by separate design codes and standards specially developed for bridge construction. The objective of the present study is to evaluate the site amplification effect on the seismic responses and behavior of bridges.

The behavior of four different types of soil: normal to lightly over consolidated clay, NC; heavily to over consolidated clay, OC; alluvial sand and silt, AS; and dense sand, DS in modifying the bedrock input ground motions are investigated in this study.

In Canada, a new approach to quantify the seismic hazard by uniform hazard spectra (UHS) has been proposed (Adams et al 1999). In this new approach, the seismic hazard at a site is specified or quantified by means of a uniform hazard spectrum curve, which is the envelope function over the frequency range of interest of the individual spectrum curves resulting from the contributing different earthquake source mechanisms. Therefore a uniform hazard spectrum gives the expected seismic hazard level at different frequencies, even though these may come from different earthquake sources but with the same probability of occurrence. Typically, the short period part of the uniform hazard spectrum is governed by contributions from moderate earthquakes from nearby sources, whereas the long period of the spectrum is governed by the contribution from large earthquakes at more distant sources. Simulated ground motions compatible with the hazard levels considered in the uniform hazard spectra (UHS) maps developed recently by the Geological Survey of Canada (Adams et al. 1999) are used as the seismic input motions in this study. Two hazard levels are provided for the uniform hazard level. The median spectral values in the uniform hazard spectra of two confidence levels, 50 percentile (median) and 84 percentile, at 10% and 2% probability of exceedance in 50 years, are considered in the present study.

Montreal is selected as the site representative in the east and Vancouver is selected for the west in this study to examine the differences of seismicity characteristics on the soil effects in modifying the ground motions.

In Canada, it is proposed that the new uniform hazard spectrum approach be adopted by the National Building Code of Canada (NBCC) as the basis for specifying the level of seismic hazard across Canada. Many investigations have been carried out or are currently underway to develop seismic design procedures which will provide a more consistent level of safety against seismic hazards across Canada. In these investigations, there are many issues that need to be addressed. These include the hazard level that should be considered in deriving the UHS, the appropriate probability and confidence level to be considered in seismic codes, the effects of foundation soil on the UHS, the representation of UHS in code provisions, and the consideration of inelastic behavior and ductility, reserve strength and how should these be accounted for in the code provisions, and finally the use of UHS in modal analysis. Most of the previous studies focus on the seismic design of buildings. The objective of the present study is to obtain information relevant to the seismic design of bridges.

The finite element computer program FLUSH is employed to evaluate the influences of different soil deposits on the free-field responses of the ground motions. The FLUSH program was developed by Lysmer et al., (1975). It considers the non-linear behavior of soil deposits subjected to strong earthquake motions by combining the equivalent linear method employed by Seed and Idriss (1969) in soil response analysis and the method of complex responses in the frequency domain. The input excitation to the finite element model in the FLUSH program is specified by an acceleration time history at the rigid bedrock, which need not be horizontal. The soil-structure interaction effects are not considered in the present study.

3.2 Soil Models

Four types of homogeneous soil deposits are considered in this study:

1. normal to lightly over consolidated clay (NC);
2. heavily over consolidated clay (OC);
3. alluvial sand and silt (AS);and
4. dense sand (DS).

The normal to lightly over consolidated clay is considered as an example of soft soil deposit, and the dense sand as an example of stiff soil deposit. The properties which effect the dynamic responses of the soil deposit are:

- mass density of soil deposit, ρ ;
- shear modulus at a shear strain level below 0.0001%, G_0 , as a function of the depth, Z ;
- secant shear modulus, G , as a function of the shear strain, γ , at the shear strain level greater than 0.0001%; and
- hysteretic damping ratio, β , as a function of the shear strain, γ .

The variation of the soil initial shear modulus, G_0 , with depth of up to 40 m below the ground surface of the soil types considered in this study is illustrated in Fig 3.1 (Elhamdi et al. 1990). The mass density, ρ , of the soil type for normal to lightly over consolidated clay is taken as 1590 kg/m^3 ; as 1937 kg/m^3 for the soil type overconsolidated clay; as 2000 kg/m^3 for alluvial sand soil type; and as 2040 kg/m^3 for dense soil type. The applied Poissons's ratio in the current study are taken as 0.45, 0.35, 0.2, 0.3 for normal to lightly over consolidated clay, heavily over consolidated clay, alluvial sand and silt and dense sand, respectively (Rahgozar, 1993)..

The study by Elhamdi and Heidebrecht (1990) shows that the effect of site amplification becomes independent with soil depth beyond 40m of uniform soil deposits. This is primarily due to the fact that as bedrock motions propagate upwards through the soil deposit, the amplification effect becomes more pronounced as the motions are influenced more by the lower shear wave velocity of the upper layer of the deposit close to the ground surface.

The relations of the ratio of secant shear modulus to initial shear modulus, G/G_0 , with the shear strain, γ , are shown in Fig 3.2, whereas that of the hysteretic damping ratio, β , with the shear strain, γ , are shown in Fig 3.3. The detailed derivations of these relationships can be found in the references by Papin et al. (1989), Heidebrecht et al. (1990) and Henderson et al. (1990).

Three soil depths 15m, 25m and 40m, are considered in the current study. The soil deposits are assumed to consist of horizontal layers. In the free field response analysis, the soil deposit is divided into finite layers. In the finite element model of the free field analysis the vertical dimension of each solid element or the thickness of each soil layer, is less than the limit of (Lysmer et al 1975) H_{max} , where

$$H_{max} = \frac{V_s}{5v_{max}} \quad (3.1)$$

where V_s is the shear wave velocity of the soil layer, and v_{max} is the highest response frequency considered in the analysis. In this study, the highest frequency of interest is taken to be 20 Hz. This limitation on the maximum height of the soil element is necessary in order to model accurately the wave propagating process of the motions through the soil layer. As the seismic wave propagates upwards through the soil deposit, the motions are influenced by the lower shear wave velocity of the upper layers of the soil deposit. If the thickness of these layers is too large, some of the high frequency components of the motions may be artificially filtered out, particularly in soft soils such as the normal to lightly over consolidated clay soil profile considered

in this study. The size of the finite element mesh employed in the present study for all soil types is established based on the requirement of the normal to lightly overconsolidated clay soil, which is the softest soil in the study with the lowest shear wave velocity. Following the procedure suggested by Lysmer et al. (1975), preliminary free-field response analyses can be performed using the computer programs SHAKE or FLUSH. These analyses related to one-dimensional soil columns that estimate the shear wave velocity that can be expected in soil deposits. The soil deposits of 15, 25 and 40m deep in normal to lightly over consolidated clay are analyzed. For the case of the soil deposit of 40m deep, the thickness of the top layer, which is the thinnest layer in the finite element model, is taken to be 1.0 m and the bottom layer, which is the thickest, is to be 2.5 m. For the case of the soil deposit depth of 25m, the thickness of the top layer is taken as 1.0 m and the bottom layer as 2 m. In the case of 15m, the thickness of the top layer is taken as 1.0 m and the bottom layer as 1.5m. The horizontal dimension of the element may be taken as a multiple of the vertical dimension (Lysmer et. al., 1975). This is because the major part of the motion consists of the vertically propagating wave with horizontal wave fronts. In the analyses, the horizontal dimension of the soil elements is taken to be 2.5 m. The width of the finite element mesh is taken for all cases to be 30m. In the third dimension perpendicular to the plane of the finite element mesh, the thickness of the soil is also taken equal to 30m. Figs 3.4 to 3.6 show the finite element meshes for the 15, 25 and 40 m soil deposit profiles. Viscous boundaries are imposed at the internal joint connections along the planar surfaces of the slices of soil in the finite element model to simulate the radiation of seismic energy away from the soil mass. In addition, the Lysmer-Waas boundaries are specified at the vertical sides of the finite element mesh to simulate the dynamic and damping or energy loss effects of the semi-infinite soil deposit beyond the finite element mesh. The soil deposits and foundations are modeled by 2-D plain strain elements in the FLUSH program.

3.3 Input Seismic Ground Motion

Since the mid 1970's, methodologies are available to derive the linear elastic spectra for a given site at specified hazard levels using the seismicity data and the ground motion attenuation relationships of the area. For a given site, a uniform hazard spectrum can be obtained by taking the envelope of the spectral acceleration response at the specified vibration period of an elastic single-degree-of-freedom (SDOF) system subjected to the earthquakes that contribute to the seismicity of the site. The UHS acceleration response spectrum is thus the envelope or the maximum function of the responses produced by different earthquakes with varying magnitudes and source distances from the site. Thus the maximum short-period spectral values may result from short-distance earthquakes, while the long-period values may be contributed by more distant earthquakes. The UHS is the preferred basis for seismic mapping when compared to spectra derived indirectly from peak ground motion bounds, because the former provide response parameters that can be used directly in estimating the design earthquake forces.

The Geological Survey of Canada has produced new seismic hazard maps of Canada based on the uniform hazard spectra described above. It has been proposed that the uniform hazard spectra be incorporated in the next revision of the National Building Code of Canada (NBCC) for the seismic design of buildings in Canada. The new seismic hazard maps incorporate the information of new earthquake data, recent research on source zones and earthquake occurrence, and research on strong ground motions relations. In the derivation of the new seismic hazard maps, pseudo-spectral acceleration, (PSA), values of 5% damped single degree-of-freedom system subjected to different source earthquakes are calculated over the range of vibration periods important for common engineering structures. These are computed assuming firm ground condition at 10% and 2% probabilities of exceedance in 50 years (1/500 and

1/2500 per annum, respectively) and at two confidence levels of 50 percentile (median) and 84 percentile. The maximum envelope of all the calculated spectral values at each vibration period gives the uniform hazard spectra (UHS). The uniform hazard spectra of population centers exposed to seismic hazards in Canada are conveniently presented in a tabulated form.

In terms of seismic hazard level, the uniform hazard spectra values with probability of exceedance of 10% in 50 years are similar to the hazard level of the 1985 seismic hazard maps. It is assumed that the current 1985 seismic hazard maps used in National Building Code of Canada, which have been formulated based on the west coast California experience, provide adequate seismic safety for western Canada in Vancouver and hence proportionately across the country. However, if other factors such as the over strength of the structural system either due to the higher material strength than the nominal strength used in the design or the contribution from the non-structural elements are considered, it is found that a structure in the west is able to resist an earthquake with considerably longer recurrence interval than the same structure located in the east (Adams et al. 1999). Therefore the use of the present methodology for quantifying the seismic hazards means that structures in the west designed in accordance with the present seismic provisions are able to survive a considerably higher hazard level than similar structures in the east. In other words the present earthquake design provisions lead to an inconsistent level of safety in the design of structures depending on their locations. To overcome this inconsistency, a more rational and uniform level of safety in earthquake resistant design across the country can be achieved by adopting the methodology, which uses a long return period earthquake as the basis of the design. The force level for the design of the structure is then obtained from the calculated forces, determined from the collapse-level earthquake with a long period, with a reduction factor which considers the expected material over strength, or the contribution from the non-structural

elements, or still other factors in the structure system. The uniform hazard spectra proposed for incorporation in the National Building Code of Canada are derived for seismic hazard with a 2500 year return period (2% probability of exceedance in 50 years).

Of the two confidence levels included in the new seismic maps of Canada, the median level estimates include the variability associated with the randomness of the physical process (aleatory uncertainty), but the uncertainty associated with the modeling of the physical process (epistemic uncertainty) is not included, whereas the 84 percentile values prescribed in the new hazard maps include both kinds of uncertainty. For regular structures, it is considered appropriate to use the median values for their design, whereas for more critical structures it is more appropriate to use the 84 percentile values, although alternative approaches are possible. Tables 3.1 and 3.2 show the elastic spectral acceleration values of 5% damped single-degree-of-freedom system for 12 cities in eastern Canada at both the 10% and 2% in 50 years probability levels. Tables 3.3 and 3.4 show the comparable values for 10 cities in western Canada.

In this study, the site amplification effects for different soil types are obtained by using time history ground motions compatible with the UHS median values. To compare with the existing 1985 seismic hazard maps and to obtain insight on the level of safety implied in the new uniform hazard spectra, seismic ground motions with 10% probability of exceedance in 50 years are used as input motions relative to the bedrock. To evaluate the new methodology of using longer return period earthquakes to achieve a more consistent level in seismic design of structures regardless of their locations in Canada, ground motions with 2% probability of exceedance in 50 years compatible with the corresponding uniform hazard spectra are also used in the site amplification study.

The numerical procedures, for deriving the simulated spectrum compatible ground motions, are described in the references by Atkinson et al (1998, 1999). Time histories of earthquakes at different combination magnitude and epicentral distance are simulated separately for eastern and western Canada. Ground motions compatible with the seismic hazards specified in the uniform hazard spectra for a number of selected cities in eastern and western Canada can be obtained from the set of simulated ground motion time histories. Using an appropriate event from the set of simulated ground motion time histories, and a fine-tune scale factor, earthquake motions that are compatible to either the short or long period part of the uniform hazard spectrum at a particular city can be obtained. These are presented in Tables 3.5 and 3.6 for the uniform hazard spectra with 10% and 2% probability of exceedance in 50 years, respectively.

In the present study, Vancouver in the west and Montreal in the east are chosen as the representative sites in western and eastern Canada for comparison. The parameters used to obtain the UHS compatible earthquake records with 10% and 2% probability of exceedance in 50 years are presented in Tables 3.7 and 3.8. The short and long period UHS spectrum compatible acceleration time-histories for these two cities are shown in Figs. 3.7 to 3.12. The target UHS and the average spectra obtained from the compatible ground motions are presented in Figs. 3.15 and 3.16 for both probability levels. The acceleration response spectra of the compatible ground motion acceleration time histories are presented in Figs. 3.18 and 3.17.

To determine the validity of the results obtained using simulated earthquake ground motions comparable with the uniform hazard spectra, actual recorded earthquake ground motions appropriate to the seismic hazard expected at the sites are also used in the analyses. In the 1985 seismic zoning maps of Canada, the seismicity of a site is specified by two parameters, the zonal velocity and the zonal acceleration.

The relative significance of the hazards generated from the two different scenarios of earthquake sources is determined by the ratio of the peak ground acceleration to peak ground velocity a/v . Ground motions with high a/v ratios are characteristics of ground motions near the epicenter of moderate to large earthquakes, whereas low a/v ratios are associated with motions occurring at large epicentral distances from large earthquakes. In Canada, studies have shown that the seismic motions in eastern Canada typically have higher a/v ratios than those in western Canada. Using the data given in the new national seismic maps and the seismicity data provided in the Appendix C of NBCC (1995), it is determined that the a/v ratios for most of the eastern locations range between 1.5 and 2.5. In western Canada, the typical a/v ratios are around 1.0, and a few locations have even lower a/v ratios with values as low as about 0.5. According to these data, the seismic hazard expected in Montreal has a high a/v ratio equal to 2, whereas the seismic hazard at Vancouver is characterized by an intermediate a/v ratio equal to 1. Therefore two sets of actual earthquakes, with high and intermediate a/v ratios, are used as input motions in the comparison study with the simulated ground motion results.

Four ground motion records are selected for each a/v ratio, as shown in Table 3.9. The acceleration time histories of the ground motions of these records are shown in Figs 3.13 and 3.14. The selected acceleration ground motions are scaled such that the peak horizontal acceleration (PHA) is the same as that specified in the seismic zoning map of the 1995 NBCC for the specific cities.

3.4 Free Field Seismic Response Analysis

As discussed before, the site amplification effect is the tendency of the soil deposit to focus the seismic energy of the bedrock motions within a narrow band of frequencies in the neighborhood of its natural frequency. To assess this behavior, free field response analysis of the soil is carried out. In the present study, a total of 384

analyses have been performed for the four different soil models using the different input ground motions by the FLUSH program. In the analysis of the results, the parameter of spectral ratio is defined here as the ratio of the acceleration response of a SDOF system located on the surface of the soil to that of the same SDOF system on the bedrock. The results of the spectral ratio versus the vibration period of the SDOF system are plotted. The influence of the soil type, soil depth and frequency content of the ground motions on the site amplification effect, are studied in the free field response analyses of the described models. The ground motions used in this part of the study are compatible with median UHS with 10% probability of exceedance in 50 years.

3.4.1 Effects of Soil Type and Depth

Figs 3.19 to 3.22 show the variation of the free field spectral ratio with the period, for the four types of soil deposit at 15m depth subjected to short and long period ground motions in Montreal and Vancouver. The results presented in Figs 3.19 show that when subjected to short periods of Montreal earthquakes, the level of amplification is similar for all four soil types considered in the study, with the dense sand amplifying the bedrock motion slightly more than the other soil types with the spectral ratio reaching 4.5. Although the peak value of the free field spectral ratio is similar, the period where the peak spectral ratio occurs is different especially for the conditions of normal consolidated clay and the dense sand, respectively having the lowest and the highest density in the group. The peak spectral ratio for the case of the dense sand occurs at around the period of 0.35g second, whereas the peak for the normal to lightly over consolidated clay occurs at about 1.3 seconds. The peak of the spectral ratio curves for the other two types of soil, overconsolidated clay and alluvial sand and silt occur between these extremes. The spectral ratio curve of normal consolidated clay in Fig 3.19 shows that between the soil deposit interval of 0.03 to 0.17 second, the soil deposit actually reduces the level of vibration at the free field

top surface compared to the bedrock motion. The period intervals of similar behavior for the other soil types narrower.

For the case of dense sand, this behavior of reducing the level of vibration is not as pronounced and the spectral ratio remains above one. The behavior of the soil deposits subjected to long period earthquakes, as shown in Fig 3.20 are similar except that the alluvial sand and silt has the highest spectral ratio. The peaks of the spectral ratios for all the soil types are higher than those obtained from short period earthquakes. The behaviors of the soil deposits in Vancouver are different from those observed in Montreal. The peaks of the spectral ratio curves for the different deposits are not the same with the normal consolidated clay soil deposit that has the highest peak spectral ratio of 4.8, and dense sand that has the lowest response, as shown in Fig 3.21. For the long period ground motions in both cities, the alluvial sand soil gives the highest peak, which indicates that this condition must amplify the ground motions the most.

Figs 3.23 to 3.26 show the variation of the free field spectral ratio with period, for the four types of soil deposit at 25m depth. Similar behaviors, as discussed for the case of 15m depth, are observed here. The short period earthquakes in Montreal and Vancouver, induce similar behavior in the spectral ratio curves for all soil types. For long period ground motions, the normal to lightly over consolidated clay soil gives the maximum spectral ratio of 5.35.

The free field spectral ratios for the case of 40m depth are presented in Figs 3.27 to 3.30. At this depth, the normal to lightly consolidated clay soil results in the highest amplification in all cases, except for the long period earthquakes in Montreal.

The results obtained here show that the vibration period of the normal to

lightly over consolidated clay soil deposit (where the peak spectral ratio occurs) is the longest among the soil types considered in the study. Dense sand has the shortest one. The amplification effect exhibited by overconsolidated clay, and alluvial sand and silt are similar. It is also significant to note that for the short period component of the motions (less than 0.1s) the spectral ratio can be less than 1.0 and as low as 0.5, especially in the case of normally consolidated clay soil, which implies that the structure in such a period range may experience less intensive seismic ground motions than the same structure located on rock sites.

To get better insight into the phenomenon of soil amplification, the calculated results are re-plotted in different groups in order to evaluate and observe more accurately the influence of the different parameters. Therefore to examine in more detail the effect of soil depth on the amplification behavior, the results presented earlier are re-plotted in Figs 3.31 to 3.46.

Figs 3.31 and 3.32 show the behavior of normal to lightly over consolidated clay soil deposit in Montreal earthquakes where the peak spectral ratio occurs at 25m depth. On the other hand, the results for the Vancouver earthquakes, as shown in Figs 3.33 and 3.34, indicate that the peak spectral ratio occurs there in the case of 40m depth.

With sites in both Montreal and Vancouver of overconsolidated clay and alluvial sand and silt soil deposits subjected to short period earthquakes, the soil depth case of 25m experience the highest amplification. For the case of overconsolidated clay soil, the highest spectral ratio occurs in the case of 15m depth, whereas for alluvial sand soil, the highest peak occurs in the case of 40m depth from long period events. For long period events, there seems to be a trend that the amplification decreases with the increase in depth for the soil deposit of overconsolidated clay and alluvial sand and silt.

The amplification behaviors for dense sand are presented in Figs 3.43 to 3.46. The case of dense sand seems also to agree with the observation that soil of 25m depth amplifies the bedrock motion more than the other cases of soil depth. It is also observed that the period corresponding to the peak spectral ratio becomes longer as the depth increases from 15m to 40m.

3.4.2 Effect of Frequency Content of Ground Motions

To investigate the effects of frequency content of the ground motions on the amplification behavior, the results presented earlier are re-plotted in different groupings in Figs 3.47 to 3.58.

The results of the four soil types at 15m depth are presented in Figs 3.47 to 3.50. The results show that under these conditions, the long period earthquakes lead to higher peak spectral ratios than the short period earthquakes with the exception of normal to lightly over consolidated clay soil subjected to Vancouver earthquakes.

Figs 3.51 to 3.54 show the results for the cases of 25m depth. The spectral ratios of sites with 25m soil are higher for long period Montreal and Vancouver earthquakes. The spectral ratio increases more than 30% for the cases of normal consolidated clay soil when they are subjected to long period events compared to the short period events.

The results of the free field spectral ratios for 40m depth of different soil deposits are shown in Figs 3.55 to 3.58. The peak spectral ratios are similar in magnitude in all cases.

The peak spectral ratios and the periods where the peaks occur are summarized

in Table 3.10. The results show that the periods where the peak spectral ratios occur are generally similar regardless whether the input motions are short or long period earthquakes.

3.5 Spectra of Different Ground Motions

As it was discussed, it is found that the level of safety can be achieved more consistently across the country by using an earthquake with higher probability level of occurrence to quantify the seismic demand and then reduced to the design level taking into consideration the effect of overstrength, which can be expected in typical structures. It is proposed that the seismic hazard at 2% probability of exceedance in 50 years can provide a more consistent level of safety compared to the current probability level of 10% in 50 years. With this new direction in defining a seismic hazard probability level, it is therefore important to evaluate the influence of this new approach on the soil amplification effects and soil factors in design code.

In this section, the soil amplification effect of the four types of soil deposits resulted from UHS compatible simulated earthquake ground motions with 10% probability of exceedance in 50 years are compared to the results obtained using the 2% probability of exceedance in 50 years. These comparisons also include the soil amplification effect of the soils deposits subjected to actual recorded earthquakes. These recorded earthquakes are selected such that their a/v ratios, which are known to be associated with frequency content of seismic motions, are comparable to the expected earthquake events at Montreal and Vancouver. The selected acceleration ground motions are scaled such that the peak horizontal acceleration (PHA) of these records is the same as that specified in the seismic zoning map of the 1995 NBCC for the specified cities.

3.5.1 Free Field Spectra

The free field spectral ratios of the four considered soil deposits subjected to actual recorded earthquakes with high a/v ratio and intermediate a/v ratio are compared to the results obtained from the short and long period simulated ground motions compatible to the uniform hazard spectra with 10% and 2% probability of exceedance in 50 years in Montreal and Vancouver. The results of the comparisons are shown in Figs 3.59 to 3.82. The peak spectral ratios and the periods when the peaks occur using the UHS compatible ground motions with 10% and 2% probability of exceedance in 50 years are presented in Tables 3.10 and 3.11. The changes in peak spectral ratios subjected to the UHS compatible simulated ground motions with 2% probability of exceedance with respect to the results obtained from the UHS compatible simulated ground motions with 10% probability of exceedance are presented in Table 3.13.

As shown in Figs 3.59 to 3.82, the free field spectral ratio curves for each of the different depths of the four types of soil deposits subjected to the UHS compatible simulated ground motions with 10% and 2% probability of exceedance in 50 years and the actual recorded earthquakes are very similar for the periods longer than 0.1 s, in both cities. The period when the peak spectral ratio of the soil deposit occurs is not effected by the severity of the seismic hazard considered in the analysis. It is also observed that using the UHS compatible simulated ground motions with longer return period (2% probability of exceedance in 50 years) can sometimes decrease the spectral ratios compared to the results obtained from the UHS compatible ground motions with 10% probability of exceedance. In fact, the results presented in Table 3.13 indicate that the peak spectral ratio for the case of 40m depth dense sand soil in Vancouver, subjected to the UHS compatible ground motions with 2% probability of exceedance is less than 20% of the result obtained from the UHS compatible ground motions with 10% probability of exceedance in 50 years. These results should be

considered when defining a soil factor in the seismic provisions, if adoption of the proposed uniform hazard spectra with 2% probability of exceedance in 50 years is approved in the future seismic codes.

3.5.2 Acceleration Response Spectra

The design spectra in most of the seismic provisions are given in terms of acceleration response spectra. The acceleration response spectra of the four types of soil deposits subjected to the UHS compatible ground motions with 10% and 2% probability of exceedance, and the recorded earthquakes are presented in Figs 3.83 to 3.106. The peak spectral acceleration and the periods when the peak occurs are presented in Tables 3.14 to 3.16.

From the Figs 3.83 to 3.106, it is observed that the shapes of the spectra are similar for all the cases using different ground motions. For the case of Montreal, when using the UHS compatible simulated ground motions with 10% probability of exceedance in 50 years, the peak spectral acceleration occurs between 0.2 s and 1.0 s. The highest peak acceleration due to soil amplification effect occurs with dense sand. In Vancouver, there are two period intervals where acceleration responses are high and the spectra show two peaks. The amplification effect occurs over a wider range of vibrational period, between 0.2 and 1.55 s. The results presented in Table 3.14 show that the peak spectral acceleration due to the soil amplification effects can be up to 6 times higher than the expected peak ground acceleration specified in the seismic provisions for both Montreal and Vancouver.

For the soil deposits subjected to UHS compatible ground motions with 2% probability of exceedance in 50 years the similar trends are observed, although the calculated acceleration responses are significantly higher than results obtained from the UHS compatible ground motions with 10% probability of exceedance in 50 years.

For instance, in Vancouver, for the case of 15m depth of the dense sand soil deposits, using short period UHS compatible ground motions with 2% probability of exceedance in 50 years an acceleration as high as 3.5g is expected to occur which is 2.5 times the calculated results obtained from using short period UHS compatible ground motions with 10% probability of exceedance in 50 years.

The response spectra of various soil deposits subjected to actual recorded earthquakes with high a/v ratios indicate that maximum acceleration for the sites considered in the present study occurs for the periods between 0.15 and 0.36 seconds. The peak accelerations are from 0.5g to 0.8g; except for the case of 15m depth dense sand soil where the peak acceleration goes up to 1.6g. For the earthquakes with intermediate a/v ratios, the response spectral accelerations are higher and the periods where the peak acceleration occur are longer compare to those of high a/v ratios.

It is also observed that the period of the peak spectral acceleration is different from the period when the peak free field spectral ratio occurs.

3.6 Conclusion

The amplification effects of four different soil types in three different depths subjected to UHS compatible time history ground motions with 10% and 2% probability of exceedance in 50 years and actual recorded earthquakes with comparable frequency content are investigated in this chapter. The following conclusions and observations can be made from the results:

1. All the soil profiles amplify the bedrock motions, but to different degrees.
2. The effect of soil amplification is not as significant if natural frequency of the soil deposit does not coincide with the frequency range where the seismic energy of the ground motion is concentrated.

3. The peaks of the amplification of normal to lightly over consolidated clay soil occur at a longer period than the other soil types, whereas that of dense sand occur at the shortest period, regardless of short and long period earthquakes.
4. The case of 25m depth of the soil deposits amplify the bedrock motion more than the cases of shallower and deeper ones, in most cases. The period at the time of peak spectral ratio becomes longer as the depth increases from 15m to 40m.
5. The dense sand soil deposits can considerably amplify the bedrock motions especially in the case of deep soil deposits.
6. The free field spectral ratio curves for each of the three depths of the four types of soil deposits subjected to the actual recorded earthquakes and the UHS compatible ground motions with 2% and 10% probability of exceedance in 50 years are similar for the periods longer than 0.1 s, in both cities.
7. The peak spectral acceleration and the peak free field spectral ratio of the soil deposits occur at two different vibrational periods for each of the different soil deposits. The period when these peaks occur are not effected by the severity of the seismic hazard considered in the analysis.

Table 3.1: Uniform hazard spectra values for Eastern Canada, 10% in 50 years robust hazard, on firm ground*.

Period (s)	St. John's	Halifax	Moncton	Fredericton	La- Malbaie	Quebec	Trios- Rivieres	Montreal	Ottawa	Niagra Falls	Toronto	Windsor
0.10	5.9	9.2	10	14	94	24	27	30	29	18	11	5.8
0.15	7.2	9.9	10	13	110	24	26	29	29	16	9.7	5.8
0.20	8.6	11	12	17	100	24	26	29	28	16	11	6.6
0.30	7.8	9.2	10	13	71	19	18	21	20	11	8.3	5.1
0.40	6.3	7.3	8.3	10	57	15	14	15	15	8.4	6.3	4.0
0.50	5.4	6.2	6.9	8.6	48	13	11	13	12	6.5	5.0	3.3
1.00	2.8	3.0	3.0	3.7	20	5.7	4.5	5.2	5.0	2.7	2.2	1.3
2.00	0.8	0.9	0.9	1.1	6.1	1.7	1.5	1.6	1.5	0.8	0.6	0.4
PGA	3.6	7.2	7.2	9.4	59	16	18	20	20	12	7.9	4.0
PGV	0.029	0.033	0.04	0.05	0.28	0.071	0.071	0.07	0.076	0.045	0.033	0.021

* Spectral (5% damped) and peak acceleration values are in %g, peak velocity in m/s.

Table 3.2: Uniform hazard spectra values for Eastern Canada, 2% in 50 years robust hazard, on firm ground*.

Period (s)	St. John's	Halifax	Moncton	Fredericton	La- Malbaie	Quebec	Trios- Rivieres	Montreal	Ottawa	Niagra Falls	Toronto	Windsor
0.10	13	20	31	39	190	58	63	68	66	46	27	16
0.15	15	22	29	36	220	61	67	71	69	41	28	16
0.20	18	23	30	39	230	59	64	69	67	41	28	17
0.30	16	19	23	29	170	43	47	50	48	31	22	13
0.40	13	15	19	24	140	34	35	39	38	25	16	10
0.50	11	13	16	20	120	29	31	34	32	20	13	8.3
1.00	6.0	7.0	6.8	8.6	59	14	12	14	14	7.1	5.4	3.9
2.00	1.6	1.9	2.1	2.7	19	4.8	4.3	4.8	4.5	2.1	1.5	1.1
PGA	9.0	12	21	27	110	37	40	43	42	30	20	12
PGV	0.57	0.071	0.095	0.12	0.62	0.16	0.17	0.18	0.18	0.13	0.081	0.055

* Spectral (5% damped) and peak acceleration values are in %g, peak velocity in m/s.

Table 3.3: Uniform hazard spectra values for Western Canada, 10% in 50 years robust hazard, on firm ground*.

Period(s)	Calgary	Kelowna	Kamloops	Prince George	Vancouver	Victoria	Tofino	Prince Rupert	Queen Charlotte	Inuvik
0.10	4.2	8.9	8.8	3.5	44	59	20	11	28	3.1
0.15	6.3	13	12	5.2	52	69	28	16	39	4.8
0.20	6.8	13	13	5.7	52	68	29	18	43	5.4
0.30	6.1	12	12	5.1	46	58	27	16	42	5.3
0.40	5.0	10	9.9	4.3	40	50	23	14	39	4.6
0.50	4.1	8.5	8.7	3.6	35	45	20	13	36	4.0
1.00	2.0	4.9	5.4	2.1	18	20	11	8.8	24	2.3
2.00	1.1	2.9	3.2	1.4	8.9	9.6	6.2	5.1	13	1.5
PGA	4.0	7.1	7.1	3.3	26	34	14	9.2	22	3.2

* Spectral (5% damped) and peak acceleration values are in %g.

Table 3.4: Uniform hazard spectra values for Western Canada, 2% in 50 years robust hazard, on firm ground*.

Period(s)	Calgary	Kelowna	Kamloops	Prince George	Vancouver	Victoria	Tofino	Prince Rupert	Queen Charlotte	Inuvik
0.10	10	18	18	8.3	83	110	43	24	43	6.1
0.15	14	26	26	12	100	120	28	34	59	9.1
0.20	15	27	28	13	100	120	29	36	65	10
0.30	13	24	25	12	87	110	27	33	66	9.2
0.40	10	20	20	9.6	76	92	23	28	64	7.8
0.50	8.4	17	17	8.0	67	83	20	24	62	6.7
1.00	4.1	8.9	10	4.0	34	38	11	16	50	3.9
2.00	2.3	5.3	6.0	2.5	18	19	6.2	9.3	26	2.5
PGA	8.8	14	14	48	48	62	26	17	35	6.0

* Spectral (5% damped) and peak acceleration values are in %g.

Table 3.5: Applicable time histories to match UHS with 10% probability of exceedance in 50 years

CITY	Short-period event	Fine-tune scale factor	Long-period event	Fine-tune scale factor
Eastern Canada				
Montreal	M5.5 at R=30	0.80	M7.0 at R=150	0.70
Ottawa	M5.5 at R=30	0.80	M7.0 at R=150	0.70
La Malbaie	M5.5 at R=15	1.20	M7.0 at R=30	0.70
Quebec City	M5.5 at R=30	0.80	M7.0 at R=150	0.70
Halifax	M5.5 at R=70	1.20	M7.0 at R=300	0.70
Toronto	M5.5 at R=70	1.20	M7.0 at R=300	0.70
Western Canada				
Vancouver	M6.0 at R=20	1.1	M7.2 at R=70	0.80
Victoria	M6.0 at R=20	1.1	M7.2 at R=70	0.80
Tofino	M6.0 at R=30	1.0	M7.2 at R=70	0.70
Prince George	M6.0 at R=70	0.70	M7.2 at R=200	0.70

Table 3.6: Applicable time histories to match UHS with 2% probability of exceedance in 50 years

CITY	Short-period event	Fine-tune scale factor	Long-period event	Fine-tune scale factor
Eastern Canada				
Montreal	M6.0 at R=30km	0.85	M7.0 at R=70km	0.90
Ottawa	M6.0 at R=30km	0.80	M7.0 at R=70km	0.80
La Malbaie	M7.0 at R=30km	0.75	M7.0 at R=20km	0.75
Quebec City	M6.0 at R=30km	0.70	M7.0 at R=70km	0.80
Halifax	M6.0 at R=50km	0.75		
Toronto	M6.0 at R=50km	0.75		
Western Canada				
Vancouver	M6.5 at R=30	1.0	M7.2 at R=70	1.0
			M8.5 Cascadia	2.2
Victoria	M6.5 at R=30	1.2	M8.5 Cascadia	2.2
Tofino	M6.5 at R=50	1.0	M8.5 Cascadia	2.2
Prince George	M6.5 at R=50	0.25	M8.5 Cascadia	2.2

Table 3.7: The characteristics of UHS with 10% probability of exceedance in 50 years compatible records (Atkinson, 1998)

Description	Duration(s)	Source distance(km)	Moment Magnitude	Scale factor	PGA (%g)
Montreal					
Short Period 1	7.82	30	5.5	0.80	17.44
Short Period 2	7.82	30	5.5	0.80	21.23
Long Period 1	24.87	150	7.0	0.70	15.93
Long Period 2	24.87	150	7.0	0.70	13.64
Vancouver					
Short Period 1	6.00	20	6.0	1.1	21.06
Short Period 2	6.00	20	6.0	1.1	22.83
Long Period 1	19.66	70	7.2	1.0	9.99
Long Period 2	19.66	70	7.2	1.0	9.99

Table 3.8: The characteristics of UHS with 2% probability of exceedance in 50 years compatible records (Atkinson, 1999)

Description	Duration(s)	Source distance(km)	Moment Magnitude	Scale factor	PGA (%g)
Montreal					
Short Period 1	8.8	30	6.0	0.85	43.03
Short Period 2	8.8	30	6.0	0.85	52.23
Short Period 3	8.8	30	6.0	0.85	57.85
Short Period 4	8.8	30	6.0	0.85	38.74
Long Period 1	24.07	70	7.0	0.90	30.10
Long Period 2	24.07	70	7.0	0.90	28.59
Long Period 3	24.07	70	7.0	0.90	34.20
Long Period 4	24.07	70	7.0	0.90	29.16
Vancouver					
Short Period 1	8.53	30	6.5	1.0	53.37
Short Period 2	8.53	30	6.5	1.0	53.70
Short Period 3	8.53	30	6.5	1.0	57.85
Short Period 4	8.53	30	6.5	1.0	38.74
Long Period 1	18.18	70	7.2	1.0	24.62
Long Period 2	18.18	70	7.2	1.0	25.93
Long Period 3	18.18	70	7.2	1.0	23.03
Long Period 4	18.18	70	7.2	1.0	25.19

Table 3.9: Selected recorded ground motion

a/v	Event Name	Year	a_{max} (m/s ²)	v_{max} (m/s)	a/v	Soil condition
High	Parkfeild	1966	0.434	0.255	1.70	Rock
	Montana	1935	0.146	0.072	2.03	Rock
	Nahanni	1985	1.101	0.462	2.38	Rock
	Seguenay	1988	0.107	0.016	7.09	Rock
Low	Long Beach	1933	0.097	0.236	0.41	Rock
	El Centro	1934	0.16	0.209	0.77	Rock
	San Fernando 3	1971	0.129	0.186	0.69	Stiff Soil
	San Fernando 4	1971	0.106	0.170	0.62	Stiff Soil

Table 3.10: Peak Spectral Ratios and the Corresponding Periods for UHS with 10% probability of exceedance in 50 years Compatible Ground Motions

	SOIL Type		15m		25m		40m	
			Short Period	Long Period	Short Period	Long Period	Short Period	Long Period
Montreal	NC	PSR	4.27	5.28	3.91	5.34	4.46	4.63
		Period(s)	1.26	1.26	1.66	1.55	2.04	1.90
	OC	PSR	3.73	5.10	4.40	5.13	4.33	4.30
		Period(s)	0.68	0.72	1.02	1.02	1.35	1.35
	AS	PSR	4.12	5.49	4.51	5.12	3.88	4.66
		Period(s)	0.73	0.73	1.02	1.02	1.55	1.45
	DS	PSR	4.50	5.24	4.51	4.95	4.15	5.44
		Period(s)	0.34	0.34	0.48	0.48	0.73	0.73
Vancouver	NC	PSR	4.81	4.55	4.09	5.38	5.06	5.88
		Period(s)	1.26	1.18	1.55	1.55	1.91	2.04
	OC	PSR	4.09	5.37	4.38	4.74	3.71	4.39
		Period(s)	.72	.68	.96	1.02	1.45	1.45
	AS	PSR	4.13	5.37	4.48	4.90	3.70	4.78
		Period(s)	.73	.68	.96	1.02	1.55	1.45
	DS	PSR	4.06	4.40	4.46	5.28	4.27	5.22
		Period(s)	.34	.34	.48	.51	.73	.68

Table 3.11: Peak Spectral Ratios and the Corresponding Periods for UHS with 2% probability of exceedance in 50 years Compatible Ground Motions

	SOIL Type		15m		25m		40m	
			Short Period	Long Period	Short Period	Long Period	Short Period	Long Period
Montreal	NC	PSR	4.13	5.21	4.51	5.45	3.57	4.79
		Period(s)	1.18	1.18	1.55	1.55	2.04	2.04
	OC	PSR	4.85	4.68	3.95	4.70	3.92	4.20
		Period(s)	.73	.68	1.02	.96	1.35	1.45
	AS	PSR	4.85	5.25	4.11	4.77	3.98	4.56
		Period(s)	.73	.73	1.02	1.02	1.55	1.45
	DS	PSR	5.16	4.67	4.58	5.17	4.81	5.22
		Period(s)	.34	.31	.48	.51	.73	.73
Vancouver	NC	PSR	5.02	5.23	5.00	5.84	3.90	5.5
		Period(s)	1.26	1.26	1.55	1.55	1.91	1.91
	OC	PSR	4.28	4.73	4.4	4.05	4.48	4.71
		Period(s)	.68	.68	1.02	1.02	1.45	1.45
	AS	PSR	4.63	4.91	4.55	4.17	4.53	4.88
		Period(s)	.73	.68	1.02	1.02	1.45	1.45
	DS	PSR	4.34	4.55	3.62	4.69	4.63	4.91
		Period(s)	.32	.34	.48	.48	.73	.73

Table 3.12: Peak Spectral Ratios and Corresponding Periods of Actual Recorded Earthquakes

	SOIL Type		15m	25m	40m
Montreal (H a/v)	NC	PSR	5.52	4.72	4.57
		Period(s)	1.26	1.45	2.04
	OC	PSR	4.58	3.68	4.31
		Period(s)	0.68	1.10	1.35
	AS	PSR	4.73	3.96	4.26
		Period(s)	0.73	1.10	1.45
	DS	PSR	4.84	4.34	4.68
		Period(s)	0.34	0.48	0.73
Vancouver (I a/v)	NC	PSR	5.26	4.97	5.46
		Period(s)	1.26	1.66	1.91
	OC	PSR	4.14	4.81	4.22
		Period(s)	.68	1.02	1.45
	AS	PSR	4.32	4.89	4.13
		Period(s)	.73	1.02	1.45
	DS	PSR	4.52	4.72	4.26
		Period(s)	.32	.48	.73

Table 3.13: The Changes of Peak Spectral Ratio Due to Using UHS 2% Instead of 10% Probability of Exceedance in 50 years ¹

	Soil Type	15m		25m		40m	
		Short Period	Long Period	Short Period	Long Period	Short Period	Long Period
Montreal	NC	-3.3	-1.3	15.3	2.1	-20.0	3.5
	OC	30.0	-8.2	-10.2	-8.4	-9.5	-2.3
	AS	17.7	-4.4	-8.9	-6.8	2.6	-2.1
	DS	14.7	-10.9	1.6	4.4	15.9	-4.0
Vancouver	NC	4.4	14.9	22.2	8.6	-22.9	-6.5
	OC	4.6	-11.9	0.5	-14.6	20.8	7.3
	AS	-4.4	24.3	-8.4	0.2	-10.4	15.7
	DS	6.9	3.4	-18.8	-11.2	8.4	-5.9

(1) The negative numbers show the decrease and the positive numbers show the increase of peak spectral ratios in percent.

Table 3.14: Peak Acceleration Response and the Corresponding Periods for UHS with 10% probability of exceedance in 50 years Compatible Ground Motions

	SOIL Type		15m		25m		40m	
			Short Period	Long Period	Short Period	Long Period	Short Period	Long Period
Montreal	NC	PSA	0.68	0.59	0.60	0.62	0.45	0.51
		Period(s)	0.28	0.34	0.28	0.68	0.28	0.83
	OC	PSA	0.70	0.82	0.48	0.50	0.39	0.35
		Period(s)	0.24	0.68	0.36	0.96	0.28	0.55
	AS	PSA	0.85	0.88	0.57	0.61	0.42	0.39
		Period(s)	0.28	0.68	0.24	0.39	0.28	0.55
	DS	PSA	0.95	1.18	0.72	0.81	0.82	0.86
		Period(s)	0.36	0.34	0.20	0.48	0.28	0.68
Vancouver	NC	PSA	1.02	0.61	0.91	0.78	0.81	0.67
		Period(s)	0.20	0.48	0.42	1.55	0.59	1.91
	OC	PSA	1.04	0.84	0.77	0.55	0.56	0.57
		Period(s)	0.22	.68	0.22	1.02	0.22	1.55
	AS	PSA	0.98	0.84	0.88	0.57	0.58	0.64
		Period(s)	0.26	0.68	0.22	1.02	0.22	1.55
	DS	PSA	1.45	0.84	1.18	0.95	0.94	0.81
		Period(s)	0.34	0.36	0.48	0.48	0.26	.68

Table 3.15: Peak Acceleration Response and the Corresponding Periods for UHS with 2% probability of exceedance in 50 years Compatible Ground Motions

	SOIL Type		15m		25m		40m	
			Short Period	Long Period	Short Period	Long Period	Short Period	Long Period
Montreal	NC	PSA	1.47	1.19	1.15	1.06	1.04	0.85
		Period(s)	0.32	0.26	0.26	0.45	0.34	0.34
	OC	PSA	1.28	1.24	1.06	0.86	0.82	0.74
		Period(s)	0.22	0.68	0.34	0.36	0.32	0.51
	AS	PSA	1.56	1.35	1.07	0.98	0.90	0.74
		Period(s)	0.28	0.26	0.34	0.24	0.32	0.36
	DS	PSA	2.88	2.16	1.48	1.79	1.50	1.29
		Period(s)	0.34	0.34	0.48	0.48	0.28	0.26
Vancouver	NC	PSA	2.01	1.33	2.06	1.54	1.81	1.00
		Period(s)	0.34	1.26	0.72	1.55	0.72	0.78
	OC	PSA	2.56	1.74	1.87	1.46	1.31	1.57
		Period(s)	0.63	0.68	1.02	0.95	1.35	1.45
	AS	PSA	2.73	1.97	1.93	1.01	1.42	1.30
		Period(s)	0.72	0.72	1.02	0.83	1.35	1.45
	DS	PSA	3.52	2.14	2.03	1.76	2.73	1.97
		Period(s)	0.34	0.34	0.48	0.48	0.72	0.72

Table 3.16: Peak Acceleration Response and the Corresponding Periods

CITY	SOIL Type		15m	25m	40
Montreal (H a/v)	NC	PSA	0.82	0.69	0.66
		Period(s)	0.36	0.36	0.36
	OC	PSA	0.52	0.69	0.44
		Period(s)	0.15	0.36	0.36
	AS	PSA	0.61	0.77	0.52
		Period(s)	0.28	0.36	0.36
	DS	PSA	1.59	0.80	0.59
		Period(s)	0.36	0.36	0.28
Vancouver (I a/v)	NC	PSA	1.45	1.32	1.20
		Period(s)	0.51	0.45	0.55
	OC	PSA	1.42	1.31	0.93
		Period(s)	0.68	0.95	0.55
	AS	PSA	1.48	1.29	1.00
		Period(s)	0.68	0.95	0.55
	DS	PSA	2.07	2.02	1.45
		Period(s)	0.34	0.48	0.68

Figure 3.1: Profile of initial shear modulus with depth for NC, OC, AS and DS Sites

Figure 3.2: Shear modulus variation with shear strain for various Soils

Figure 3.3: Damping ratio variation with shear strain for various soils

Figure 3.4: Finite element model of 15m deep soil deposit on the bedrock

Figure 3.5: Finite element model of 25m deep soil deposit on the bedrock

Figure 3.6: Finite element model of 40m deep soil deposit on the bedrock

Figure 3.7: Spectrum-Compatible time history acceleration for 10%/50 year UHS used for Montreal

Figure 3.8: Spectrum-Compatible time history acceleration for 10%/50 year. UHS used for Vancouver

Figure 3.9: Spectrum-Compatible time history acceleration for 2%/50 year UHS used for Montreal, short period (M6.0 at R=30km)

Figure 3.10: Spectrum-Compatible time history acceleration for 2%/50 year UHS used for Montreal, long period (M7.0 at R=70km)

Figure 3.11: Spectrum-Compatible time history acceleration for 2%/50 year UHS used for Vancouver, short period (M6.5 at R=30km)

Figure 3.12: Spectrum-Compatible time history acceleration for 2%/50 year UHS used for Vancouver, long period (M7.2 at R=70km)

Figure 3.13: Time History of Selected Ground Motion Records with High a/v Ratios

Figure 3.14: Time History of Selected Ground Motion Records with Intermediate a/v Ratios

Figure 3.15: Target 10%/50 year UHS from GSC for Montreal and spectra of the selected compatible time histories

Figure 3.16: Target 10%/50 year UHS from GSC for Vancouver and spectra of the selected compatible time histories

Figure 3.17: Response acceleration spectra of the UHS compatible records used for Montreal for 10%/50 year & 2%/50 year

Figure 3.18: Response acceleration spectra of the UHS compatible records used for Vancouver for 10%/50 year & 2%/50 year

Figure 3.19: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Montreal (depth=15m)

Figure 3.20: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Montreal (depth=15m)

Figure 3.21: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Vancouver (depth=15m)

Figure 3.22: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Vancouver (depth=15)

Figure 3.23: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Montreal (depth=25m)

Figure 3.24: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Montreal (depth=25m)

Figure 3.25: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Vancouver (depth=25m)

Figure 3.26: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Vancouver (depth=25m)

Figure 3.27: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Montreal (depth=40m)

Figure 3.28: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Montreal (depth=40m)

Figure 3.29: Free field spectral ratio of various soil deposits subjected to short period compatible ground motions in Vancouver (depth=40m)

Figure 3.30: Free field spectral ratio of various soil deposits subjected to long period compatible ground motions in Vancouver (depth=40m)

Figure 3.31: Free field spectral ratio of NC soil deposits subjected to short period 10%/50 year compatible ground motions in Montreal

Figure 3.32: Free field spectral ratio of NC soil deposits subjected to long period 10%/50 year compatible ground motions in Montreal

Figure 3.33: Free field spectral ratio of NC soil deposits subjected to short period 10%/50 year compatible ground motions in Vancouver

Figure 3.34: Free field spectral ratio of NC soil deposits subjected to long period 10%/50 year compatible ground motions in Vancouver

Figure 3.35: Free field spectral ratio of OC soil deposits subjected to short period 10%/50 year compatible ground motions in Montreal

Figure 3.36: Free field spectral ratio of OC soil deposits subjected to long period 10%/50 year compatible ground motions in Montreal

Figure 3.37: Free field spectral ratio of OC soil deposits subjected to short period 10%/50 year compatible ground motions in Vancouver

Figure 3.38: Free field spectral ratio of OC soil deposits subjected to long period 10%/50 year compatible ground motions in Vancouver

Figure 3.39: Free field spectral ratio of AS soil deposits subjected to short period 10%/50 year compatible ground motions in Montreal

Figure 3.40: Free field spectral ratio of AS soil deposits subjected to long period 10%/50 year compatible ground motions in Montreal

Figure 3.41: Free field spectral ratio of AS soil deposits subjected to short period 10%/50 year compatible ground motions in Vancouver

Figure 3.42: Free field spectral ratio of AS soil deposits subjected to long period 10%/50 year compatible ground motions in Vancouver

Figure 3.43: Free field spectral ratio of DS soil deposits subjected to short period 10%/50 year compatible ground motions in Montreal

Figure 3.44: Free field spectral ratio of DS soil deposits subjected to long period 10%/50 year compatible ground motions in Montreal

Figure 3.45: Free field spectral ratio of DS soil deposits subjected to short period 10%/50 year compatible ground motions in Vancouver

Figure 3.46: Free field spectral ratio of DS soil deposits subjected to long period 10%/50 year compatible ground motions in Vancouver

Figure 3.47: Free field spectral ratio of 15m deep, NC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.48: Free field spectral ratio of 15m deep, OC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.49: Free field spectral ratio of 15m deep, AS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.50: Free field spectral ratio of 15m deep, DS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.51: Free field spectral ratio of 25m deep, NC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.52: Free field spectral ratio of 25m deep, OC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.53: Free field spectral ratio of 25m deep, AS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.54: Free field spectral ratio of 25m deep, DS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.55: Free field spectral ratio of 40m deep, NC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.56: Free field spectral ratio of 40m deep, OC soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.57: Free field spectral ratio of 40m deep, AS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.58: Free field spectral ratio of 40m deep, DS soil deposits subjected to 10%/50 year UHS compatible ground motions

Figure 3.59: Free field spectral ratio of 15m deep, NC soil deposits subjected to various ground motions

Figure 3.60: Free field spectral ratio of 15m deep, NC soil deposits subjected to various ground motions

Figure 3.61: Free field spectral ratio of 15m deep, OC soil deposits subjected to various ground motions

Figure 3.62: Free field spectral ratio of 15m deep, OC soil deposits subjected to various ground motions

Figure 3.63: Free field spectral ratio of 15m deep, AS soil deposits subjected to various ground motions

Figure 3.64: Free field spectral ratio of 15m deep, AS soil deposits subjected to various ground motions

Figure 3.65: Free field spectral ratio of 15m deep, DS soil deposits subjected to various ground motions

Figure 3.66: Free field spectral ratio of 15m deep, DS soil deposits subjected to various ground motions

Figure 3.67: Free field spectral ratio of 25m deep, NC soil deposits subjected to various ground motions

Figure 3.68: Free field spectral ratio of 25m deep, NC soil deposits subjected to various ground motions

Figure 3.69: Free field spectral ratio of 25m deep, OC soil deposits subjected to various ground motions

Figure 3.70: Free field spectral ratio of 25m deep, OC soil deposits subjected to various ground motions

Figure 3.71: Free field spectral ratio of 25m deep, AS soil deposits subjected to various ground motions

Figure 3.72: Free field spectral ratio of 25m deep, AS soil deposits subjected to various ground motions

Figure 3.73: Free field spectral ratio of 25m deep, DS soil deposits subjected to various ground motions

Figure 3.74: Free field spectral ratio of 25m deep, DS soil deposits subjected to various ground motions

Figure 3.75: Free field spectral ratio of 40m deep, NC soil deposits subjected to various ground motions

Figure 3.76: Free field spectral ratio of 40m deep, NC soil deposits subjected to various ground motions

Figure 3.77: Free field spectral ratio of 40m deep, OC soil deposits subjected to various ground motions

Figure 3.78: Free field spectral ratio of 40m deep, OC soil deposits subjected to various ground motions

Figure 3.79: Free field spectral ratio of 40m deep, AS soil deposits subjected to various ground motions

Figure 3.80: Free field spectral ratio of 40m deep, AS soil deposits subjected to various ground motions

Figure 3.81: Free field spectral ratio of 40m deep, DS soil deposits subjected to various ground motions

Figure 3.82: Free field spectral ratio of 40m deep, DS soil deposits subjected to various ground motions

Figure 3.83: Acceleration response spectra of 15m deep, NC soil deposits subjected to various ground motions

Figure 3.84: Acceleration response spectra of 15m deep, NC soil deposits subjected to various ground motions

Figure 3.85: Acceleration response spectra of 15m deep, OC soil deposits subjected to various ground motions

Figure 3.86: Acceleration response spectra of 15m deep, OC soil deposits subjected to various ground motions

Figure 3.87: Acceleration response spectra of 15m deep, AS soil deposits subjected to various ground motions

Figure 3.88: Acceleration response spectra of 15m deep, AS soil deposits subjected to various ground motions

Figure 3.89: Acceleration response spectra of 15m deep, DS soil deposits subjected to various ground motions

Figure 3.90: Acceleration response spectra of 15m deep, DS soil deposits subjected to various ground motions

Figure 3.91: Acceleration response spectra of 25m deep, NC soil deposits subjected to various ground motions

Figure 3.92: Acceleration response spectra of 25m deep, NC soil deposits subjected to various ground motions

Figure 3.93: Acceleration response spectra of 25m deep, OC soil deposits subjected to various ground motions

Figure 3.94: Acceleration response spectra of 25m deep, OC soil deposits subjected to various ground motions

Figure 3.95: Acceleration response spectra of 25m deep, AS soil deposits subjected to various ground motions

Figure 3.96: Acceleration response spectra of 25m deep, AS soil deposits subjected to various ground motions

Figure 3.97: Acceleration response spectra of 25m deep, DS soil deposits subjected to various ground motions

DS Soil Deposit
15m deep
Vancouver

Figure 3.98: Acceleration response spectra of 25m deep, DS soil deposits subjected to various ground motions

Figure 3.99: Acceleration response spectra of 40m deep, NC soil deposits subjected to various ground motions

Figure 3.100: Acceleration response spectra of 40m deep, NC soil deposits subjected to various ground motions

Figure 3.101: Acceleration response spectra of 40m deep, OC soil deposits subjected to various ground motions

Figure 3.102: Acceleration response spectra of 40m deep, OC soil deposits subjected to various ground motions

Figure 3.103: Acceleration response spectra of 40m deep, AS soil deposits subjected to various ground motions

Figure 3.104: Acceleration response spectra of 40m deep, AS soil deposits subjected to various ground motions

Figure 3.105: Acceleration response spectra of 40m deep, DS soil deposits subjected to various ground motions

Figure 3.106: Acceleration Response Spectra of 40m Deep, DS Soil Deposits Subjected to Various Ground Motions

Chapter 4

Response Spectra of Design Code Provisions

4.1 Introduction

Over the past three decades, there has been an impressive growth in the knowledge of earthquake engineering, including the development of the seismic provisions in design codes. However, because of the uncertainty that is inherent in the unpredictable nature of future earthquakes, there still needs improvement on the protection against property loss and life safety in the provisions of the current seismic codes.

The basic philosophy of seismic design is the same for all structures. However, there are important differences in the design requirements between bridges and those of buildings. The first generation bridge design codes did not account for these differences. The same seismic design procedure that was used for building design was adopted for the design of bridges in the bridge design codes such as AASHTO (1965). The rather poor performance of bridges during the 1971 San Fernando earthquake in southern California clearly showed that the seismic design procedures applicable to buildings could not be directly applied to bridges. As a result the state-of-the-art practices of the earthquake resistant design of bridges have undergone significant changes and major modifications have taken place in the bridge design codes in many countries.

Three generations of seismic hazard maps for Canada have been produced at

roughly 15-year intervals (1953, 1970 and 1985). The Geological Survey of Canada (GSC) has recently developed new national seismic hazard maps (Adams et al. 1999). These new maps incorporate the new information from a significant increment of earthquake data, recent research on source zones and earthquake occurrence, together with complementary research on strong motion relations. It is proposed that these new maps be adopted as the seismological basis in the future edition of the National Building Code of Canada (NBCC) (Atkinson et al. 1998). The current seismic coefficient and the design procedure considered in the proposed Canadian Highway Bridge Design Code (CHBDC, 1998) are adapted from the 1997 American Association of State Highway and Transportation Officials Load and Resistance Factor Design (AASHTO LRFD 1997).

In the view of the newly developed uniform hazard spectra and other advances in earthquake engineering, there is the need to study the impact on the seismic design practices of bridges. Studies are needed to evaluate the adequacy of the current code provisions to provide safe and adequate seismic design of bridges; determine issues related to the seismological differences between eastern and western Canada; evaluate the impact of using the UHS approach in seismic design of bridges; and study the effect of the changes of defining earthquake hazards at the probability level of 2% probability of exceedance in 50 years instead of 10% in 50 years.

The present study investigates the adequacy of the design code procedures. The design response spectra adopted in the five design codes are compared and their adequacies are evaluated.

The five seismic codes used in this study are: (1) Standard Specification

for Highway Bridges by the American Association of State Highway and Transport Officials (AASHTO, 1997); (2) the proposed Canadian Highway Bridges Design Code (CHBDC, 1998); (3) the Bridge Design Specification of California Department of Transportation (CALTRAN, 1999); (4) Ontario Highway Bridge Design Code (OHBDC, 1991); and (5) Transit New Zealand (TNZ, 1995).

The effects of four different soil types on the design requirements of bridges are considered in the present study. The selected soil types are normally to lightly over consolidated clay, *NC*; heavily to over consolidated clay, *OC*; alluvial sand and silt, *AS*; and dense sand, *DS*. The normally to over consolidated clay is an example of soft soil deposits and the dense sand is an example of stiff soil deposits. Three soil depths of 15m, 25m and 40m are considered in the present study. The width of the finite element mesh is taken to be 30m for all cases. In the third direction, perpendicular to the plane of the finite element mesh, the thickness of soil is also taken equal to 30m. These soil deposits were assumed to be composed of horizontal layers with infinite length. The response spectra of sites located on rock are also included in the comparison. Details of the soil properties and the soil models considered in the study are presented in Chapter 3.

The finite element computer program FLUSH is employed to evaluate the influences of different soil deposits on the free-field responses of the ground motions. The FLUSH computer program considers the non-linear behavior of soil deposits subjected to strong earthquake motions by combining the equivalent linear method employed by Seed and Idriss (1969) in soil response analysis, and the method of complex responses in the frequency domain. The input excitation to the finite element model in the FLUSH program is specified by an acceleration time history at the rigid bedrock which does not have to be horizontal.

The new seismic hazard maps of Canada have been produced based on the uniform hazard spectra by Geological Survey of Canada. It has been proposed that the uniform hazard spectra be incorporated in the next revision of the National Building Code of Canada (NBCC) for the seismic design of buildings in Canada. The new seismic hazard maps incorporate the information of new earthquake data, recent research on source zones and earthquake occurrence, and research on relations for strong ground motions. In the derivation of the new seismic hazard maps, pseudo-spectral acceleration, PSA, values of 5% damped single degree-of-freedom system subjected to different source earthquakes are calculated over the range of vibration periods important for common engineering structures. These are computed assuming firm soil conditions and at 10% and 2% probabilities of exceedance in 50 years ($1/500$ and $1/2500$ per annum, respectively) and at two confidence levels of 50 percentile (median) and 84 percentile. The maximum envelope of all the calculated spectral values at each vibration period gives the uniform hazard spectra (UHS). In the present study, Montreal in east and Vancouver in the west are chosen as the representative sites in western and eastern Canada for comparison.

The response spectra of different soil deposits subjected to UHS compatible ground motions with 10% in 50 years probability of exceedance are compared to the elastic response spectra considered in the seismic codes. The compatibility and adequacy of the calculated response spectra using the simulated ground motions are compared to the spectra of recorded earthquakes. The influences and significances of the differences in the characteristics of the earthquake motions in the east and in the west of Canada are investigated.

4.2 Seismic Code Provisions

The relevant seismic design code provisions of the design codes selected for the comparison study are briefly summarized here. The assumptions for the different parameters used in the seismic provisions are briefly described in the following sections. The definition of the parameters and notations are presented in Chapter 2.

4.2.1 AASHTO LRFD, 1997

The elastic seismic coefficient for the period of the m^{th} mode of vibration, T_m , in the range between 0.0 and 4.0 seconds is defined in this code as:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A \quad (4.1)$$

In the AASHTO code, the variation of the elastic seismic coefficient C_{sm} with the period of vibration includes the influences of the soil effect. The parameters used in this equation and the assumptions made for the analytical model are described in the paragraphs below.

The acceleration coefficient, A , in Eq. 4.1 represents the seismic risk at the location of the design bridge. In the present study, this coefficient is considered as 0.2 for Montreal and 0.26 for Vancouver. These are the peak ground accelerations expected from earthquakes with 10% probability of exceedance in 50 years at Montreal and Vancouver, as specified in NBCC for seismic zones 4 and 5 respectively. The normalized elastic seismic response coefficient given in the AASHTO code is multiplied by these accelerations ratios, A , to obtain the elastic response spectra for the comparison.

The site effect coefficient, S , depends on the soil properties, ranging from 1.0 for rock or stiff soil (soil profile type I) to 2.0 for soft soil (soil profile type IV). Based on the soil categorization in this code, the three different depths of normal to lightly over consolidated clay soil deposits are assumed as soil type IV. The dense sand soil

profile with 15m depth is considered type II, and 25 and 40m depths of these soil deposits are taken as soil type I. Depths of 15, 25 and 40m of the heavily overconsolidated clay and alluvial sand and silt soil profiles are identified as soil type III, II and I respectively.

The corresponding load combination factor including earthquake in this code is specified as one.

4.2.2 CHBDC, 1998

The seismic response coefficient in the proposed CHBDC (1998) code is the same as the AASHTO (1997) except that the importance factor is also included in the formula of the seismic coefficient. Thus for the period of the m^{th} mode of vibration in the range between 0.0 and 4.0 seconds, the elastic seismic response coefficient is specified as:

$$C_{sm} = \frac{1.2ASI}{T_m^{2/3}} \leq 2.5AI \quad (4.2)$$

The definitions and assumptions for different parameters in Eq. 4.2 are the same as those in Eq. 4.1 in the 1997 AASHTO code. In the comparison study, a consistent performance level must be selected from the different codes so that a valid comparison can be made. The bridge category of other bridges is considered here with the importance factor equal to 1.0.

4.2.3 CALTRAN, 1999

In the CALTRAN design manual, the elastic response spectra of a site for the maximum credible event(s) are determined as the product of the peak rock acceleration, A , the acceleration spectra in rock, R , and the soil amplification factor, S . The horizontal mean spectral acceleration can be selected from an ARS curve presented in this code for different soil conditions and earthquake magnitude. The definitions

of these parameters are provided in Chapter 2 of the present study. The necessary assumptions for the parameters defining the elastic spectra are presented in the paragraphs below.

In the comparison study, comparable values are selected to define the seismic hazard of Montreal and Vancouver in the CALTRAN procedure. The spectrum compatible ground motion time history for short and long period earthquakes in eastern and western Canada can be obtained from different magnitude-distance combinations. For Montreal the short period event is specified as an earthquake of magnitude 6 (M6.0) at 30km distance, and for the long period event, it is of magnitude 7.0 (M7.0) at 70km distance. For Vancouver the short period event is defined as an earthquake of magnitude 6.5 (M6.5) at 30km and it is of magnitude 7.2 (M7.2) at 70km for long period events (Atkinson et al. 1998, 1999). The CALTRAN design code is specifically developed for the design of bridges in California, and the differences of the east and west coast earthquakes are not included in CALTRAN code provisions. Therefore, the specified magnitude-distance combinations defining short and long period earthquakes for Montreal and Vancouver are assumed to be equivalent to the magnitude groups of 6.5 ± 0.25 and 7.25 ± 0.25 in the CALTRAN code, respectively. The peak ground acceleration values of 0.2g for Montreal and 0.26g for Vancouver with linear interpolation are used to select the appropriate ARS curves in the CALTRAN code.

In the CALTRAN code the Site modification factor, S , is developed for the six soil types (A-F), which are specified based on factors such as: shear wave velocity, standard penetration resistance, undrained shear strength and plastic index. In this study the dense sand soil deposits in all three depths, as well as the case of 40m depth of the alluvial sand and silt soil deposit are considered as equivalent to the soil type C in CALTRAN code provisions. The 15 and 25m depth of alluvial sand and silt and overconsolidated clay soil deposits are assumed to be equivalent to soil type D. The

overconsolidated clay soil deposit with 40m depth and normal to lightly consolidated clay soil deposits with 15 and 25m depths are considered equivalent to soil type E and the case of 40m depth normal to lightly consolidated clay soil deposit equivalent to soil type F.

The corresponding group load combination coefficient including earthquake in the CALTRAN code is specified as 1.0.

4.2.4 OHBDC, 1991

The seismic design spectra are presented in the Ontario Highway Bridge Design Code (OHBDC, 1991) in the form of curves of response coefficient factor C . The effects of seismicity of the site and the underlying soil as well as flexibility of the bridge are taken into account in these curves. The assumptions made in this study for different parameters of this seismic code are described in the following paragraphs.

In the present study the frame coefficient, F , is assumed to be 1.0, this is equivalent to the case in the 1991 OHBDC code that single columns or piers resist the horizontal loads.

The zonal velocity ratio for Montreal is specified as 0.1 and for Vancouver as 0.2 in the National Building Code of Canada. The response acceleration spectra are taken to be equivalent to the response coefficient curve, C , which can be obtained by using the zonal velocity of the selected cities of Montreal and Vancouver.

The cases of 15 and 25m depths of all soil deposits are assigned as equivalent to soil type 2 in the OHBDC code; and the case of the 40m depth of all the soil deposits as equivalent to the type 3.

The acceleration response spectra are scaled by 1.3 to account for the effect of the earthquake coefficient in the load combination specified in OHBDC.

4.2.5 TNZ, 1995

The spectra of the analytical results are compared to the design inelastic response spectrum, C_μ , for the specific soil conditions in the Transit New Zealand code considering a ductility of one to give the elastic spectra and factored by the zone factor Z and the risk factor R .

The TNZ code defines three importance categories for bridges depending on the average number of vehicles per day expected at the time of design, or location (motor ways or railways), or whether on a national or a provincial highway. The bridge is considered to carry or cross motorways or railway (Importance Category 1), therefore the relevant Risk Factor, 1.3, is used in the scaling of the spectra in the comparison study.

The zonal factor in TNZ, is given in the form of a contour map for the New Zealand island. In the comparison study, the maximum and minimum zonal factors in the contour map of New Zealand and the maximum and minimum peak ground acceleration specified in uniform hazard spectra for Canada are used in linear interpolation to estimate the zonal factor for Montreal and Vancouver. The zonal factors are found to be 0.85 and 1.1 for Montreal and Vancouver, respectively.

Three subsoil categories are defined in the New Zealand code based on the low amplitude natural period, unconfined compressive strength of the soil and depth of soil. Each of these subsoil categories are assigned a structural performance factor, S_P . The values for S_P are 0.90, 0.80 and 0.67 respectively. In the present study, the normal to lightly consolidated clay soil deposits in all depths as well as the case of

the 15m depth of overconsolidated clay and alluvial sand and silt soil deposits are specified as equivalent to soil type 2 in the TNZ code provisions. All other soil profiles are taken as equivalent to soil type 1. Based on this assumption, the response spectra provided for each of these soil categories are multiplied by the corresponding values of S_P (0.8 for soil type 2 and 0.9 for soil type 3).

The load combination group including earthquake in the TNZ code, defines a factor of one for earthquake.

4.3 Input Seismic Ground Motion

In the present study, the uniform hazard spectra (UHS) compatible simulated ground motions with 10% probability of exceedance in 50 years are used as input motions at the bedrock in the analysis. The 10% probability level is chosen because the spectra of all bridge design codes are derived based on this probability level. Details about UHS and the simulated ground motions considered in the study are presented in Chapter 3.

Vancouver and Montreal are chosen as the representative sites in western and eastern Canada for comparison. To determine the validity of the results obtained using simulated earthquake ground motions comparable with the uniform hazard spectra, actual recorded earthquake ground motions appropriate to the seismic hazard expected at the sites are also used in the analyses. In the 1985 seismic zoning maps of Canada, the seismicity of a site is specified by two parameters, the zonal velocity and the zonal acceleration. The relative significance of the hazards generated from the two different scenarios of earthquake sources is determined by the ratio of the peak ground acceleration to peak ground velocity a/v . Ground motions with high a/v ratios are characteristics of ground motions near the epicenter of moderate to large earthquakes, whereas low a/v ratios are associated with motions occurring at

large epicentral distances from large earthquakes. In Canada, studies have shown that the seismic motions in eastern Canada typically have higher a/v ratios than those in western Canada. Using the data given in the new national seismic maps and the seismicity data provided in Appendix C of NBCC (1995), it is determined that the a/v ratios for most of the eastern locations range between 1.5 and 2.5. In western Canada, the typical a/v ratios are around 1.0, and a few locations have even lower a/v ratios with values as low as about 0.5. According to these data, the seismic hazard expected in Montreal has a high a/v ratio equal to 2, whereas the seismic hazard at Vancouver is characterized by an intermediate a/v ratio equal to 1. Therefore two sets of actual earthquakes, with high and intermediate a/v ratios, are used as input motions in the comparison study with the simulated ground motion results. The selected acceleration ground motions are scaled such that the peak horizontal acceleration (PHA) is the same as that specified in the seismic zoning map of the 1995 NBCC for the specific cities.

4.4 Comparison of Design Codes Response Spectra with 10% UHS

The response spectra of dynamic analyses of different soil deposits subjected to 10% UHS compatible ground motions in Montreal and Vancouver are compared to the design spectra in the seismic codes. Figs 4.1 to 4.52 show the results of the comparisons.

4.4.1 Rock

Figs 4.1 to 4.4 show the results of the response spectra calculated for a bedrock site subjected to short and long period earthquakes compatible with UHS in Montreal and Vancouver with those of AASHTO (1997), CHBDC (1998), CALTRAN (1999), OHBDC (1991) and TNZ (1995) seismic codes. It is observed that for the rock sites only the spectra of the OHBDC are lower than the uniform hazard spectra with 10% probability of exceedance in 50 years for periods shorter than 1.0 s in Montreal and

p-periods shorter than 1.35 s in Vancouver earthquakes.

4.4.2 NC Soil Deposits

The comparisons between acceleration response spectra of normal to lightly overconsolidated clay soil deposits subjected to UHS compatible ground motions for Montreal and Vancouver are presented in Figs 4.5 to 4.16.

Figs 4.5 and 4.6 show the comparison between response spectra for the case of 15m depth of normal to lightly over consolidated clay soil deposits subjected to earthquakes in Montreal, and the response spectra presented in different codes. It is observed that the spectra from CALTRAN and TNZ codes are significantly higher for the structures located on this soil deposit. The AASHTO spectrum is close to the spectra of analytical results for the periods longer than 0.5s. The design spectrum of OHBDC is only adequate compared to analytical results for the periods longer than 1.3s and 1.6s of short and long period events, respectively. The same trend is observed from Figs 4.7 to 4.10 for the cases of 25m and 40m depth of this soil deposit.

Figs 4.11 to 4.16 present the comparison between the design spectra of seismic codes with the response spectra of normal to lightly overconsolidated clay soil deposits subjected to earthquakes in Vancouver. It is observed that the soil amplifications are very pronounced especially for the short period events in this city. The design spectra of the seismic codes are significantly lower than the analytical results. The spectra presented in CALTRAN and TNZ codes, are closer to the analytical results.

The acceleration response spectra of the long period events, on the other hand, show reasonable compatibility with the spectra presented in AASHTO, CHBDC, CALTRAN and TNZ design codes except for the periods close to the period of peak acceleration. The spectra of the OHBDC code is significantly lower than the spectra

obtained from the analytical results for periods longer than 0.1 s.

4.4.3 OC Soil Deposits

The acceleration response spectra of the heavily overconsolidated clay soil subjected to the UHS compatible ground motions with 10% probability of exceedance in 50 years are compared to the elastic spectra of the seismic provisions of the bridge design codes considered in the comparison study, as shown in Figs 4.17 to 4.28.

For the case of 15m depth soil deposits in Montreal, shown in Fig 4.17 and 4.18, the elastic spectra of the CALTRAN and TNZ codes are significantly higher than the analytical results especially for the periods longer than 1.0 s. The spectra presented in AASHTO and CHBDC design codes are also higher than analytical results for long periods. The design spectrum in the OHBDC code is only adequate for the periods longer than 1.2 s. The design spectra of the CALTRAN and TNZ codes are overly conservative for the cases of 25m and 40m depth soil deposits subjected to earthquakes expected to occur in Montreal, as shown in Figs 4.19 to 4.22. For these depths, the AASHTO code presents spectra closer to the spectra of the analytical results. The design spectra in the OHBDC code are significantly lower than the spectra obtained from the analytical results.

The peak spectral acceleration is very pronounced when the heavily over consolidated clay soils are subjected to short period earthquakes in Vancouver. The design spectra of all the seismic codes are inadequate for the periods close to the peak spectral acceleration, as shown in Figs 4.23 to 4.28. For the long period earthquakes in this city the same trends of behavior are observed but the peak acceleration is not as high as short period events. Consequently, the spectra of the analytical results are not very different from the spectra presented in seismic codes. The spectra presented in OHBDC code do not agree with the spectra of the analytical results.

4.4.4 AS Soil Deposits

The design spectra of different codes are compared to the response spectra of alluvial sand and silt soil deposits subjected to the UHS compatible ground motions with 10% probability of exceedance in 50 years, as shown in Figs 4.29 to 4.40.

It is observed that in Montreal the design spectra of the CALTRAN code are the closest spectra to the analytical results compared to the spectra of other bridge codes, as presented in Figs 4.29 to 4.34. The design spectra of TNZ code are higher than the spectra of the analytical results, particularly for periods longer than 1.5s. The design spectra of the AASHTO code, are higher than the spectra of analytical results for the cases of 25 and 40m depths alluvial sand and silt soil but for the case of 15m depth of soil the spectrum presented in the AASHTO code is lower than the response spectra for the periods close to the peak spectral acceleration. The design spectra of OHBDC code are only similar to the spectra of analytical results for periods longer than 1.25s.

The result of the comparisons between the design spectra provided in the seismic provisions of bridge codes considered in the present study, and the response spectra of the alluvial sand and silt soil subjected to the expected earthquakes in Vancouver are shown in Figs 4.35 to 4.40. It is observed that the design spectra of the codes are lower than the spectra of the analytical results over the short period range of the spectra, where soil amplification effects are significantly pronounced. The design spectra of the CALTRAN and TNZ seismic codes are close to the analytical results for the longer periods. The spectra of AASHTO and CHBDC are close to the analytical results for earthquakes with long period but they are lower than the response spectra for the short period events. The design spectra of OHBDC code are lower than the response spectra obtained for alluvial sand and silt soil.

4.4.5 DS Soil Deposit

The design spectra of the codes considered in the study are compared to the response spectra of dense sand soil deposits subjected to the UHS compatible ground motions with 10% probability of exceedance in 50 years, as shown in Figs 4.41 to 4.52.

Figs 4.41 to 4.46 show the comparisons of the response spectra of analytical results with the spectra of the expected earthquakes in Montreal. It is observed that for the periods shorter than 0.5 s, where significant soil amplifications occur at the dense sand soil deposits, the design spectra of the seismic code do not agree with the analytical results. The presented spectra in the AASHTO and CHBDC codes are close to the response spectra of the analytical results for the periods longer than 0.5 s. The response spectra of the analytical results are significantly lower than the design spectra of CALTRAN and TNZ codes for periods longer than 1.0 s.

The comparison of the spectra of expected earthquakes in Vancouver to the response spectra of the results are shown in Figs 4.47 to 4.52. For the case of 15m depth dense sand, as shown in Fig 4.41, it is observed that the peak spectral acceleration of the results when the soil deposits are subjected to short period events, which occurs at the periods shorter than 0.5 s goes up to 2g. Such a high acceleration is not accounted for in the design spectrum of any of the considered seismic codes. For the periods longer than 0.5 s, the design spectra of the AASHTO, CHBDC, CALTRAN, and TNZ code are close to the spectra of results obtain from the analysis. The same trend of behavior is observed for the other cases of dense sand soil deposit but the peak spectral acceleration is not as pronounced.

The design spectra presented in the OHBDC code again are significantly lower than the spectra of analytical results at both cities.

4.5 Comparison of Design Codes Response Spectra with Spectra of Actual Earthquakes

To determine the validity of the results obtained using simulated earthquake ground motions comparable with the uniform hazard spectra, actual recorded earthquake ground motions appropriate to the seismic hazard expected at the sites are also used in the analyses. The acceleration response spectra of the four types of soil deposits subjected to the actual recorded earthquakes are developed. These recorded earthquakes are selected such that their a/v ratios, which are known to be associated with frequency content of seismic motions, are comparable to the expected earthquake at Montreal and Vancouver. The selected acceleration ground motions are scaled such that the peak horizontal acceleration (PHA) of these records is the same as that specified in the seismic zoning map of the 1995 NBCC for the considered cities. The acceleration spectra resulting from the analyses are compared to the design spectra of the bridge codes considered, as shown in 4.53 to 4.76.

Figs 4.53 to 4.58 show the comparisons for the normal to lightly over consolidated clay soil deposits. It is observed that the spectra presented in the TNZ and CALTRAN codes are close to the analytical results. The design spectra in the bridge codes considered in the study are closer to the spectra of the analytical results when the normal to lightly over consolidated clay soil deposits are subjected to the high a/v ratio earthquakes than when subjected to they are subjected to earthquakes with intermediate a/v ratios.

The response spectra of the heavily overconsolidated clay soil deposits subjected to earthquake ground motions with high and intermediate a/v ratios are compared to the design spectra of the codes considered in the comparison, as shown in Figs 4.59 to 4.64. The spectra presented in the AASHTO, CHBDC, CALTRAN and

TNZ codes are similar to the spectra of the analytical results, in Montreal. In Vancouver, the design spectra given in these codes are significantly lower than the spectra of the results using earthquakes with intermediate a/v ratios.

The results of the comparisons between the design spectra of the bridge codes and the spectra of the alluvial sand and silt soil deposits subjected to high and intermediate a/v ratios are shown in Figs 4.65 to 4.70. The results are close to the spectra of the overconsolidated clay.

The design spectra of codes considered in the comparison study are compared to the spectra of the results obtained from subjecting dense sand soil deposits to the earthquakes with high and intermediate a/v ratios, as shown in Figs 4.71 to 4.76. It is observed that except for the periods close to the peak acceleration, when the peak response acceleration can go up to twice as much as the corresponding acceleration specified in the codes, the codes present compatible spectra to the analytical results.

4.6 Conclusion

The acceleration response spectra of the four different soil types in three different depths subjected to the uniform hazard spectra compatible ground motions with 10% probability of exceedance in 50 years and actual recorded earthquakes are compared to the corresponding seismic spectra of AASHTO (1997), CHBDC (1998), CALTRAN (1999), OHBDC (1991) and TNZ (1995) in this chapter. The following conclusions can be made.

1. The design spectra of all the design codes are lower than the results over some of the resonant periods.
2. The shape of the acceleration response spectra in Montreal and Vancouver are different, therefore specifying the same design spectra for eastern and western of Canada may not be reasonable. The results show that the peak spectral

acceleration is not an adequate parameter to consider the different seismic characteristics in the east and west of Canada.

3. The response spectra of the sites located in Montreal (eastern Canada) are close to the design spectra of the seismic codes considered in the present study, except for the spectra presented in the OHBDC code.
4. For Vancouver (western Canada), the design spectra need to be modified in order to take into account the seismic characteristics of the western part of the country.
5. The design spectra presented in TNZ code are higher than the results for most of the vibrational periods, but they are observed to be unnecessarily higher than the results for the long periods.
6. The design spectra of the CALTRAN seismic provisions are the closest ones , comparing to the spectra of the other bridge codes, to the spectra of analytical results.
7. The spectra presented in OHBDC code, for almost all of the cases considered have significantly lower values than the results. One reason for this may be because the design spectra in this code incorporate the effects of the ductility and other reduction factors. Using these reduced spectra means that all of the bridge components are assigned the same unknown reduction factors, which is known to be inadequate for bridge structures.
8. The parameters that contribute to soil amplification are: soil type, soil depth and the characteristics of the ground motions. In order to develop feasible design spectra to be used in the seismic provisions of the design codes all of these factors shall be accounted for.

9. Further studies need to be done to investigate the adequacy of the seismic provisions for the locations with low a/v frequency content. The results of these studies should also be included in the future seismic provisions.

Figure 4.1: Comparison of design spectra of the codes to response spectra of bedrock sites subjected to short period Montreal-related ground motions

Figure 4.2: Comparison of design spectra of the codes to response spectra of bedrock sites subjected to long period Montreal-related ground motions

Figure 4.3: Comparison of design spectra of the codes to response spectra of bedrock sites subjected to short period Vancouver-related ground motions

Figure 4.4: Comparison of design spectra of the codes to response spectra of bedrock sites subjected to long period Vancouver-related ground motions

Figure 4.5: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Montreal-related ground motions (depth=15m)

Figure 4.6: Comparison of design code response spectra to response spectra of NC soil deposits subjected to long period Montreal-related ground motions (depth=15m)

Figure 4.7: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Montreal-related ground motions (depth=25m)

Figure 4.8: Comparison of design code response spectra to response spectra of NC soil deposits subjected to long period Montreal-related ground motions(depth=25m)

Figure 4.9: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Montreal-related ground motions (depth=40m)

Figure 4.10: Comparison of design code response spectra to response spectra of NC soil deposits subjected to long period Montreal-related ground motions(depth=40m)

Figure 4.11: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions(depth=15m)

Figure 4.12: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions (depth=15m)

Figure 4.13: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions (depth=25m)

Figure 4.14: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions(depth=25m)

Figure 4.15: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions(depth=40m)

Figure 4.16: Comparison of design code response spectra to response spectra of NC soil deposits subjected to short period Vancouver-related ground motions(depth=40m)

Figure 4.17: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Montreal-related ground motions (depth=15m)

Figure 4.18: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to long period Montreal-related ground motions (depth=15m)

Figure 4.19: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Montreal-related ground motions (depth=25m)

Figure 4.20: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to long period Montreal-related ground motions(depth=25m)

Figure 4.21: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Montreal-related ground motions (depth=40m)

Figure 4.22: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to long period Montreal-related ground motions (depth=40m)

Figure 4.23: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions (depth=15m)

Figure 4.24: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions (depth=15m)

Figure 4.25: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions(depth=25m)

Figure 4.26: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions(depth=25m)

Figure 4.27: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions (depth=40m)

Figure 4.28: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to short period Vancouver-related ground motions(depth=40m)

Figure 4.29: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Montreal-related ground motions (depth=15m)

Figure 4.30: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to long period Montreal-related ground motions(depth=15m)

Figure 4.31: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Montreal-related ground motions (depth=25m)

Figure 4.32: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to long period Montreal-related ground motions (depth=25m)

Figure 4.33: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Montreal-related ground motions (depth=40m)

Figure 4.34: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to long period Montreal-related ground motions(depth=40m)

Figure 4.35: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions(depth=15m)

Figure 4.36: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions(depth=15m)

Figure 4.37: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions (depth=25m)

Figure 4.38: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions (depth=25m)

Figure 4.39: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions (depth=40m)

Figure 4.40: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to short period Vancouver-related ground motions(depth=40m)

Figure 4.41: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Montreal-related ground motions (depth=15m)

Figure 4.42: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to long period Montreal-related ground motions (depth=15m)

Figure 4.43: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Montreal-related ground motions (depth=25m)

Figure 4.44: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to long period Montreal-related ground motions(depth=25m)

Figure 4.45: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Montreal-related ground motions (depth=40m)

Figure 4.46: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to long period Montreal-related ground motions(depth=40m)

Figure 4.47: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions(depth=15m)

Figure 4.48: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions (depth=15m)

Figure 4.49: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions (depth=25m)

Figure 4.50: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions(depth=25m)

Figure 4.51: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions (depth=40m)

Figure 4.52: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to short period Vancouver-related ground motions(depth=40m)

Figure 4.53: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to H a/v ground motions (depth=15m)

Figure 4.54: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to H a/v ground motions (depth=25m)

Figure 4.55: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to H a/v ground motions (depth=40m)

Figure 4.56: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to I a/v ground motions (depth=15m)

Figure 4.57: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to I a/v ground motions (depth=25m)

Figure 4.58: Comparison of design spectra of the codes to response spectra of NC soil deposits subjected to I a/v ground motions (depth=40m)

Figure 4.59: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to H a/v ground motions (depth=15m)

Figure 4.60: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to H a/v ground motions (depth=25m)

Figure 4.61: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to H a/v ground motions (depth=40m)

Figure 4.62: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to I a/v ground motions(depth=15m)

Figure 4.63: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to I a/v ground motions (depth=25m)

Figure 4.64: Comparison of design spectra of the codes to response spectra of OC soil deposits subjected to I a/v ground motions(depth=40m)

Figure 4.65: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to H a/v ground motions (depth=15m)

Figure 4.66: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to H a/v ground motions (depth=25m)

Figure 4.67: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to H a/v ground motions (depth=40m)

Figure 4.68: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to I a/v ground motions (depth=15m)

Figure 4.69: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to I a/v ground motions(depth=25m)

Figure 4.70: Comparison of design spectra of the codes to response spectra of AS soil deposits subjected to I a/v ground motions(depth=40m)

Figure 4.71: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to H a/v ground motions (depth=15m)

Figure 4.72: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to H a/v ground motions (depth=25m)

Figure 4.73: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to H a/v ground motions (depth=40m)

Figure 4.74: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to I a/v ground motions(depth=15m)

Figure 4.75: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to I a/v ground motions (depth=25m)

Figure 4.76: Comparison of design spectra of the codes to response spectra of DS soil deposits subjected to I a/v ground motions (depth=40m)

Chapter 5

Ductility Demand of Bridge Systems

5.1 Introduction

Prior to the San Fernando earthquake of Feb 9, 1971 in southern California, experience and information on bridge damage resulting directly from seismically induced vibrational effects were limited. The behavior and performance of bridge structures during strong earthquakes were not as well understood as they are today. For instance, the reports from the Alaska earthquake of March 27, 1964 indicate that nearly all of the damage to bridges during that earthquake was caused by failure of the substructures resulting from large ground displacements, settlements, and loss of bearing capacity. The damage to the superstructures was primarily caused by foundation failures, not by seismically induced vibrational effects. The same failure pattern was also observed in nine major earthquakes in Japan prior to the 1971 San Fernando earthquake. The earthquake damage to bridges in California that occurred during the period from 1933 to 1971 (magnitudes 5.4 to 7.7) prior to the San Fernando earthquake, was mostly minor damage which was easily repairable. (Gates, 1976).

The experience on the performance of highway bridges, detailed and constructed according to the latest design codes and practices at the time, was significantly different during the 1971 San Fernando earthquake (magnitude 6.6). The total cost of the damage was 6.5 million US dollar. Many major bridges suffered

severe damage or collapsed because of the vibrational effects during that earthquake. Similar types of bridge damage and failure modes were observed during the 1987 Whittier Narrows earthquake, the 1989 Loma Prieta earthquake and the 1994 Northridge earthquake in California. The studies of the damage to bridges caused by these earthquakes indicate that column failure was a major cause of collapse in many bridges. Most of the failures were observed to have occurred in bridges that were designed in the 1960s and early 1970s using the elastic theory. The deficiencies inherent in the columns of many of these older bridges have been found to be a lack of concrete confinement in the flexural plastic hinge zone, leading to a deterioration of the concrete core and buckling of the longitudinal reinforcement; insufficient development length for longitudinal reinforcement, causing flexural capacity deterioration and bar pull out; and inadequate shear strength, leading to non ductile response. In other words the columns were completely inadequate flexural members.

In the subsequent research after the 1971 San Fernando earthquake the causes of the observed bridge damage were investigated and the concept of ductile design was further developed as the design approach against major earthquakes. The approach of ductile design has been adopted to be the basis of the seismic provisions in many design codes. A structure designed to exhibit inelastic ductile behavior relies on the dissipation of the seismic energy to avoid total collapse during the strong shaking of a major earthquake. In the ductile seismic design of bridges with column piers, the design and detailing of the structural members are carried out to allow the dissipation of the seismic energy through the formation of plastic hinges in the ductile column members during earthquake responses. To ensure the formation of the plastic hinges, the column members are designed to have a reduced elastic moment capacity determined from the elastic demand reduced by a response modification factor, R .

Ideally in ductile design, the earthquake energy is dissipated in a ductile manner, and brittle failure modes such as shear failure are avoided. However, during large earthquakes individual bridge components such as shear keys and bearings, may behave and fail in a non-ductile fashion. Consequently, the use of a uniform reduction factor in the design of all the bridge components may result in some members and components being under designed. This is a concern in consideration of the vulnerability to earthquake damage of the group of existing bridges designed after the 1971 San Fernando earthquake using the methodology of ductile design but before it was fully understood. To evaluate the seismic risk of existing bridges, it is therefore important to determine the required ductility in the bridge structure and components to resist the expected seismic actions and compare those to the available ductility. The available ductility may be determined either experimentally or analytically. The seismic codes specify the upper limit of the available ductility in the design that can be relied on through the use of the response modification factor. The response modification factor depends on the component type and, in the case of reinforced concrete columns, also depends on the detailing and confinement of the concrete in the column design.

The required ductility or the ductility demand is governed by the severity of the earthquake force on the plastic hinge of a ductile component such as a bridge column or pier. The ductility demand depends on many factors which include the seismic force level used for design; the characteristics of the earthquake, including the duration and frequency content of the earthquake motions; the characteristics of the structural system, including stiffness, mass and damping; and the location and characteristics of the plastic hinge formed.

Nonlinear time-history analysis is usually the most accurate method to determine the ductility demand of a bridge structure during large earthquakes when the

response of the bridge is expected to behave inelastically. On the other hand, elastic analysis is usually employed to obtain an estimate of the relative ductility demands among the various components.

In the present study, the displacement ductility demand, the member end rotational ductility demand, and the curvature ductility demand of two example bridges are investigated. The bridges studied are two-span bridges, one with a single reinforced concrete column and the other with a reinforced concrete pier wall. The bridges are taken to be in Montreal and Vancouver on rock sites. The ground motion time-histories used as input in the analysis are compatible with the median values of uniform hazard spectra developed by the Geological Survey of Canada with 10% probability of exceedance in 50 years. The dynamic analysis is carried out using the NEABS bridge computer analysis program.

The NEABS (Nonlinear Earthquake Analysis of Bridge Systems) computer program was originally developed by Tseng and Penzien in 1973 for nonlinear dynamic analyses of long, multi-span bridge systems. The program has the capability to carry out the dynamic time-history analysis of discrete parameter systems subjected to applied dynamic loadings and/or prescribed support motions. Bridge systems can be modeled by a combination of linear and nonlinear elements.

In the present study, the displacement ductility demand of the studied bridges on three different soil deposits is also investigated. The effects of soil amplification are considered for two different depths of each of the soil types. The three types of soil considered in the study are normal to lightly over consolidated clay, *NC*; alluvial sand and silt, *AS*; and dense sand *DS*. The behavior of over consolidated clay, *OC*, soil because of its similarity with alluvial sand and silt, *AS*, soil deposit is not considered in this Chapter. The normal to over consolidated clay is an example of soft

soil deposits and the dense sand is an example of stiff soil deposits. Two depths of 15m and 40m of the soil deposits are considered in the current study to account for shallow and deep soils. Details of the soil properties and soil models considered in the present study are presented in Chapter 3.

The finite element computer program FLUSH is employed to evaluate the influences of different soil deposits on the free-field responses of the ground motions. The FLUSH program was developed by Lysmer et al., (1975). It considers the non-linear behavior of soil deposits subjected to strong earthquake motions by combining the equivalent linear method employed by Seed and Idriss (1969) in soil response analysis and the method of complex responses in the frequency domain. The input excitation to the finite element model in the FLUSH program is specified by an acceleration time history at the rigid bedrock, which need not be horizontal. The soil-structure interaction effects are not considered in the present study. The free field output ground motions from the FLUSH computer program is used as input in the bridge response analysis by the computer program NEABS.

The displacement ductility demand of the bridges on bedrock is also obtained using the UHS compatible ground motions with 2% probability of exceedance in 50 years to explore the changes in ductility demand that may occur in case of adopting this probability level instead of 10% in 50 years probability level in the new bridge design codes.

It is expected that the results of these studies will help to achieve a better understanding of bridge behavior under earthquake motions and thus may lead to safer bridge design.

5.2 Ductility Factors

In ductile seismic design, the ductile response behavior of the bridge is usually characterized by the parameter ductility, which may be defined in terms of induced curvature, member end rotation and displacement. Under monotonic loading, a correlation exists between curvature, rotation and displacement, provided the loading and geometry of the member are defined.

5.2.1 Displacement Ductility

The displacement ductility factor, μ_Δ , is a measure of the maximum ductility required of a bridge during an earthquake. For a simple structure comprised of a single-column bent supported on a rigid foundation, the ductility demand can be calculated by the following expression:

$$\mu_\Delta = \frac{\Delta_{max}}{\Delta_y} \quad (5.1)$$

where:

μ_Δ = displacement ductility

Δ_{max} = maximum horizontal displacement at the center of mass of the superstructure resulting from earthquake support motions.

Δ_y = yield displacement

The design yield force F_y is established for design purposes by reducing the elastic earthquake response force by a response modification factor, R . Assuming the elastic and inelastic systems reach equal maximum displacement during the earthquake, the relationship between the ductility demand and the response modification factor may be determined, as shown in Fig 5.1, as follows:

$$R = \mu_\Delta = \frac{F_{max}}{F_y} \quad (5.2)$$

for structures with a medium to long period of vibration. For structures in the intermediate period range, the relationship between the ductility demand and response modification factor may be more appropriately determined using the assumption of equal maximum potential energy. The elastic strain energy (the area under the load-displacement curve) is set equal to the elasto-plastic energy under the bilinear curve of the inelastic system, as shown in Fig 5.1b. The R factor in this case is related to the ductility given in the following expression

$$R = \sqrt{2\mu_{\Delta} - 1} \quad (5.3)$$

From Eqs. 5.2 and 5.3, it can be found that the equal maximum potential energy assumption leads to larger displacement ductility demand, μ_{Δ} , than the equal maximum displacement assumption which means larger displacements in short period structures.

5.2.2 Curvature Ductility

For monotonic loading a relationship exists between the curvature ductility demand and the displacement ductility demand. The bridge bent shown in Fig 5.2 is assumed to be supported on a rigid foundation. An assumed horizontal seismic force is applied at the center of gravity of the superstructure. Assuming a relatively flexible superstructure that does not significantly interact with the column, the maximum displacement at the center of the mass when the column yields at the base is:

$$\Delta_y = \frac{PL^3}{3EI} = \frac{M_y L^2}{3EI} \quad (5.4)$$

where M_y is the column moment at first yield, EI is the flexural rigidity of the column section, and L is the height of the column.

The curvature at the column base at first yield is given by:

$$\phi_y = \frac{M_y}{EI} \quad (5.5)$$

Thus Eq. 5.4 may be rewritten as:

$$\Delta_y = \frac{\phi_y L^2}{3} \quad (5.6)$$

Additional displacement causes the formation of a plastic hinge, which occurs over an equivalent plastic hinge length, h . If the plastic hinge rotation is given by θ_p , the plastic hinge rotation may be expressed in terms of curvature as follows

$$\theta_p = (\theta_{max} - \theta_y)h \quad (5.7)$$

Assuming that the rotation in the plastic hinge occurs at one end of the member, the total displacement of the superstructure may be written as follows

$$\Delta_{max} = \Delta_y + \theta_p L \quad (5.8)$$

for elastic-perfectly plastic behavior of the hinge. The ductility may be expressed as follows

$$\mu_\Delta = \frac{\Delta_y + \theta_p L}{\Delta_y} \quad (5.9)$$

Substituting Eqs 5.6 and 5.8 into Eq. 5.9 yields

$$\mu_\Delta = 1 + \frac{3h}{L} \left(\frac{\phi_{max}}{\phi_y} - 1 \right) \quad (5.10)$$

Thus the required maximum curvature ductility may be expressed in terms of the displacement ductility as follows

$$\mu_\phi = \frac{\phi_{max}}{\phi_y} = 1 + \frac{L}{3h} (\mu_\Delta - 1) \quad (5.11)$$

for the bridge column shown in Fig 5.2.

5.2.3 Rotational Ductility Demand

Maximum rotational ductility demands, μ_θ , may be obtained from the calculated maximum rotation θ_{max} and the yield rotation θ_y as follows

$$\mu_\theta = \frac{\theta_{max}}{\theta_y} \quad (5.12)$$

where:

$$\theta_y = h\phi_y = \frac{hM_y}{EI} \quad (5.13)$$

The results of nonlinear time history analyses, however, give more realistic results than the above theoretical estimates of the ductility factors.

5.3 Modeling of Bridge Systems

5.3.1 Bridge 1-Single Column Bridge

A two-span reinforced concrete box-girder bridge with a single column is analyzed. The structural elevation and plan of the bridge are shown in Fig 5.3. This bridge is analyzed by the computer program NEABS (Imbsen and Penzien, 1986).

The bridge is supported on pile foundation at the abutments and on a spread footing on Bent 2. The cross section of the column is 5ft by 7ft. For the analysis model, the column end on the foundation at the bent is assumed to be fixed. At the abutment supports, translation is allowed only in the longitudinal direction due to the shear key. Rotation is allowed about the transverse axis at the abutments. The force-displacement relationship of the energy-absorbing device oriented in the longitudinal direction of the bridge is shown in Figs 5.3b and 5.3c. A lumped mass model is used to idealize the bridge, as shown in Fig 5.4a. Three different element types are used to model the bridge. The bridge is modeled as an equivalent beam using beam column elements. Boundary element is used to model the foundation conditions at abutments. Hinge connection is assumed at the connection of the structure to the bents. The cross sectional properties of the structural members are presented in Table 5.1.

The material properties of the members are as follows:

Young's modulus of concrete= $432000 \frac{\text{kips}}{\text{ft}}^2$;

Poisson's ratio=0.18;

28-day compressive stress= $3.0 \frac{\text{kips}^2}{\text{in}}$;

and the mass density= $0.004658 \frac{\text{kips-sec}^2}{\text{ft}}$; and

for steel bar $E = 29000 \frac{\text{kips}^2}{\text{in}}$; and

$F_y = 40 \frac{\text{kips}}{\text{in}}$

As shown in Fig 5.4c, the bridge column is modeled by three members, two nonlinear members at the ends and an elastic member in the middle. The length of the middle member is twice the length of the nonlinear members, which model the plastic hinge zone of the column. The two nonlinear end members may be modeled as acting in single curvature, which is shown in Fig 5.4d. The yield surface of the nonlinear member is presented in Fig 5.5. The ultimate strengths of the column are determined as follows:

$P_u = 17640$ kips;

$M_{yu} = 9308$ kips-ft;

$M_{zu} = 12870$ kips-ft; and

$P_t = 4318$ kips

An elastic spring having a stiffness coefficient of 272,700 kips/ft in the horizontal direction is used to represent the restraint actions of the abutment backwall and piles.

The structural damping is assumed proportional to mass and stiffness and taken equal to 5%. The first 30 mode shapes are determined for the bridge. The first ten periods are listed in Table 5.2. Based on the first and tenth mode, the Rayleigh's damping coefficients are $\alpha = 0.885$ and $\beta = 0.00136$.

From Eq. 5.4 the yield displacement is found to be 0.0515in in the longitudinal direction and 0.0691in in transverse direction. The yield rotation is calculated to be 0.0009 rad about the longitudinal direction and 0.00092 rad about the transverse direction using Eq. 5.13.

5.3.2 Bridge 2- Blackstone River Bridge (Pier Wall)

The two-span Blackstone River Bridge with a concrete deck on steel girder, located on Liard highway in Northwest Territories, is analyzed. Each span has a length of 40.3m. The elevation, a typical cross section of the bridge and the finite element model are shown in Fig 5.6. The bridge deck and the steel girders and the bridge piers are modeled by 3D beam-column elements. The cross sectional properties of structural members are presented in Table 5.3.

The material properties of the members are as follows:

Young's modulus of concrete = 25 MPa;

Possion's ratio = 0.18;

28-day compressive stress = 25, 000 MPa;

Mass density = 2400 kg/m³;

and for steel:

E=200,000 MPa;

$f_y = 400$ MPa;

Possion's ratio = 0.295; and

$\rho = 7800kg/m^3$

The support conditions of the bridge are roller supports at the abutments and a fixed end support at the bridge pier.

As shown in the Fig 5.6, the 8m long column is modeled using three members, two 816mm long nonlinear members at the ends and an elastic member in the center. The length of the nonlinear member in modeling plastic hinge zone of the pier is taken as the minimum of the amounts suggested by Paulay and Priestley (1993) and those given in the CALTRAN standard code. According to the study by Paulay and Priestly (1993), the length of the plastic hinge zone in a cantilever wall is of the order:

$$l_p = (0.20 + 0.044A_r)l_w \quad (5.14)$$

where:

l_p = length of the plastic hinge;

l_w = horizontal length of the wall section; and

A_r = aspect ratio of full height of the cantilever wall to its length l_w .

The bridge is supposed to have elastic behavior along the longitudinal direction. In the CALTRAN code [9] the length of the plastic region of a pier wall can be estimated in the weak direction by the expression established for columns as follows

$$L_p = \begin{cases} 0.08L + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} & \text{US customary} \\ 0.08L + 0.022f_{ye}d_{bl} \geq 0.044f_{ye}d_{bl} & \text{SI units} \end{cases} \quad (5.15)$$

where:

L_p = equivalent analytical plastic hinge length (ft, m);

L = member length from the point of maximum moment to the point of contra-flexure (ft, m);

d_{bl} = nominal bar diameter of longitudinal column reinforcement (in, mm); and

f_{ye} = expected yield stress (ksi, MPa).

From Eq. 5.14 the plastic hinge length is found to be 2000mm and from the Eq. 5.15 this amount is found to be 816mm. The latter is used in the analyses as the length of the nonlinear member.

The nonlinear members are assumed to act in single curvature. The bridge pier wall is assumed to be unconfined (Saatcioglu and Razvi 1992). The calculated ultimate strengths of the wall is:

$$P_u = 146,800,000 \text{ N};$$

$$M_{yu} = 6,330,000 \text{ N.m};$$

$$M_{zu} = 47,630,000 \text{ N.m}; \text{ and}$$

$$P_t = 6,100,000 \text{ N}.$$

The structural damping is taken to be 5%. The first 30 mode shapes are determined for the bridge. The first ten periods are listed in Table 5.4. Based on the period of the first and the tenth modes, Rayleigh's damping constants, proportional to mass and stiffness, are $\alpha = 1.089$ and $\beta = 0.001101$ (Clough and Penzien 1975).

From the Eqs. 5.4 and 5.13 the yield displacement and yield rotation are found to be 0.0134 m and 0.0005 rad along the transverse axis. It is expected that the pier wall will have an elastic behavior along the longitudinal direction.

5.4 Analytical Results

5.4.1 Ductility Demands

The maximum displacement (Δ_{max}) of Bridge 1 at the top of the column in the longitudinal and transverse directions, and at the top of the pier wall in transverse direction of Bridge 2, and the maximum end rotation θ_{max} are calculated using

the NEABS computer program for the bridge systems located in Montreal and Vancouver. The displacement ductility, curvature ductility and member end rotation ductility demands are also estimated by means of Eqs. 5.1, 5.11 and 5.12. These results are tabulated in Tables 5.5 to 5.7.

From the results presented in Tables 5.5 and 5.6, it is observed that, the single column bridge behaves elastically under most cases, except for the short period earthquake 1 in Vancouver. In fact, along the longitudinal direction only about the 25% of the elastic capacity of the bridge is utilized to resist the seismic action, according to the displacement ductility demand, when the bridge is located in Montreal.

Table 5.7 shows similar behavior of the Blackstone Bridge with pier wall. The displacement ductility demands, however, are higher in this case.

The rotational ductility demands exert up to four times greater results than the displacement ductility demands. The curvature ductility demands are based on displacement ductility demands, as expressed by Eq. 5.11, therefore, they are only feasible when the maximum displacement is greater than yield displacement ($\mu_{\Delta} \geq 1.0$).

The ductility demands are found to be higher for the bridges located in Vancouver, especially for the short period events compared to the same results in Montreal. These facts should be considered when defining a response reduction factor to account for the ductility in the bridge elements.

5.4.2 The Effect of Underlying Soil on Ductility Demand

The displacement ductility demands are calculated for the two bridges considered in the present study on different soil profiles with two different depths. The

results are summarized in the Tables 5.8 to 5.10.

Table 5.8 shows the displacement ductility demands for the bridges located on 15 and 40m in depth of normal to lightly over consolidated clay soil deposits in Montreal and Vancouver. It is observed that the ductility demand increases, up to four times in some cases, with respect to the ductility demands on rock. The single column bridge-1 behaves elastically in both longitudinal and transverse directions when subjected to the ground motions of Montreal. For the case of Vancouver the bridge behaves inelastically and plastic hinges are formed. The pier wall of Blackstone Bridge exhibits plastic behavior in the longitudinal direction when subjected to Montreal and Vancouver earthquakes. The displacement ductility demand ranges between 2.5 to 4.5 times these corresponding demands when the bridges are located on rock. The displacement ductility demand increases as the depth of the soil deposit increases for both bridges. Smaller displacement ductility demands are obtained when the bridges are subjected to short period earthquakes in Montreal, whereas in Vancouver the displacement ductility demands of long period earthquakes are higher than those of short period ones.

The maximum displacement and the ductility demand for the specified bridges located on 15 and 40m depth alluvial sand and silt soil deposits are calculated and shown in Table 5.9. Similar results as normal to lightly over consolidated clay soil are observed for the alluvial sand and silt soil deposits.

The behaviors of these bridges are different when they are located on dense sand soil deposits with 15 and 40m depths. In most of these cases the ductility demand decreases as the depth increases. The maximum displacements in both longitudinal and transverse directions of Bridge 1 (single column) and longitudinal direction of Bridge 2 (pier wall) when subjected to Montreal earthquakes are lower than the

corresponding values for the alluvial sand and silt, and the normal to lightly over consolidated clay soil deposits, in some cases as low as half of these values. Subjected to compatible ground motions in Vancouver, the soil deposits cause larger displacements at the top of the bridge column or pier in the case of dense sand than the corresponding results for alluvial sand and silt, and normal to lightly overconsolidated clay soil deposits.

5.4.3 The Effect of Using UHS Compatible Ground Motions with 2% in 50 years Probability Level on Ductility Demand

The maximum top displacement of column in longitudinal and transverse direction of Bridge 1 and maximum top displacement of pier wall in longitudinal direction for Bridge 2 subjected to the UHS compatible ground motions with 2% probability of exceedance in 50 years for the bridges founded on rock in Montreal and Vancouver are evaluated using the NEABS computer program. The results of these analyses are presented in Table 5.11.

It is observed that the displacements of Bridge 1 (single column) in Montreal remain in elastic region in both directions. Nonlinear behavior is exerted by Bridge 2 in both cities, as well as, in both directions of Bridge 1 in Vancouver. In Montreal, the average ductility demand caused by long period events is greater than the one from short periods, whereas in Vancouver the opposite behavior is observed in all cases. The average ductility demands for short and long period events in both cities are larger compare to the relevant results from the 10% probability of exceedance in 50 years.

5.5 Conclusion

The nonlinear behavior of two bridge systems founded on rock and three different soil types subjected to different seismic loads are found using the NEABS computer program. The following conclusions can be made.

1. The rotational ductility demands, μ_θ , in terms of values are larger than displacement ductility demand, μ_Δ .
2. The curvature ductility demands, μ_ϕ give meaningful values only when the bridge starts to behave inelastically ($\mu_\Delta \geq 1.0$).
3. The displacement ductility demands of the bridge founded on the soil types considered in the present study are larger in comparison to the displacement ductility demand of the bridge founded on rock.
4. The displacement ductility demands increase when the depth of underlying soil deposits increase from 15 to 40m for bridges founded on the normal to lightly over consolidated clay and alluvial sand and silt soil deposits. On the other hand, increasing the depth of the dense sand soil deposits leads to smaller displacement ductility demands in bridges.
5. In Montreal, the average displacement ductility demand caused by long period events is larger than those from the short period ones. This is in contrast with the results obtained in Vancouver, where the larger displacement ductility demands belongs to short period earthquakes.
6. Larger displacements are expected to occur when the uniform hazard spectra with 2% probability of exceedance in 50 years level is considered compared to the results from 10% probability of exceedance in 50 years.
7. Based on the results of the present study and considering the differences between seismic characteristics in the east and west of Canada, the possibility of

recommending different response reduction factors for structures in eastern and western parts of Canada can be considered in the future seismic provisions.

8. More studies are needed in order to investigate the behavior of multi-span bridges with structural properties different from the considered cases in the present study, under different seismic actions. More experimental and analytical work is also required to obtain more information about the characteristics and location of the plastic hinges in the different components of the bridge substructure. The ductility demand of other bridge components such as connections should also be studied.

Table 5.1: The Properties of the Member Cross Section of Bridge 1

Location	$A_x(ft^2)$	$I_x(ft^4)$	$I_y(ft^4)$	$I_z(ft^4)$
Superstructure	86.0	862.0	360.0	13000
Column	35.0	215.8	72.9	142.9

Table 5.2: Structural Periods(s) of Bridge 1

Mode	1	2	3	4	5	6	7	8	9	10
Period	0.61	0.573	0.383	0.308	0.196	0.155	0.135	0.160	0.0993	0.093

Table 5.3: The Properties of the Member Cross Section of Bridge 2

No.	Material Type	$A_x(m^2)$	$I_x(m^4)$	$I_y(m^4)$	$I_z(m^4)$
1	Steel	0.090	0.0071	0.0118	0.0376
2	Steel	0.070	0.0070	0.01	0.016
3	Steel	0.113	0.0071	0.0338	0.0385
4	Steel	0.090	0.0068	0.0284	0.0146
5	Steel	0.136	0.0134	0.0661	0.0393
6	Steel	0.104	0.0129	0.0548	0.009
7	Steel	0.138	0.0167	0.078	0.0393
8	Steel	0.103	0.0162	0.065	0.0076
9	Steel	0.090	0.001	0.011	0.0376
10	Steel	0.027	0.001	0.0121	0.0004
11	Steel	0.041	0.001	0.0244	0.0008
12	Steel	0.041	0.001	0.0375	0.0008
13	Concrete	7.060	1.42	0.370	45.94

Table 5.4: Structural Periods of Bridge 2 (s)

Mode	1	2	3	4	5	6	7	8	9	10
Period	0.506	0.387	0.224	0.217	0.189	0.176	0.154	0.099	0.0826	0.0816

Table 5.5: Ductility Demands, Bridge 1, X-dir

Description	$\Delta_{max} * 10^{-2}$ (m)	$\theta_{max} * 10^{-4}$ (rad)	μ_{Δ}	μ_{ϕ}	μ_{θ}
Montreal					
Short Period, Earthquake 1	1.12	4.21	0.217	-	0.47
Short Period, Earthquake 2	1.29	3.83	0.25	0.0	0.43
Long Period, Earthquake 1	1.38	4.07	0.269	0.03	0.45
Long Period, Earthquake 2	1.47	4.16	0.285	0.5	0.46
Vancouver					
Short Period, Earthquake 1	6.18	17.2	1.201	1.27	1.91
Short Period, Earthquake 2	4.09	12.3	0.794	0.73	1.37
Long Period, Earthquake 1	1.74	4.47	0.337	0.12	0.50
Long Period, Earthquake 2	1.47	4.54	0.284	0.05	0.50

Table 5.6: Ductility Demands, Bridge 1, Y-dir

Description	$\Delta_{max} * 10^{-2}$ (m)	$\theta_{max} * 10^{-4}$ (rad)	μ_{Δ}	μ_{ϕ}	μ_{θ}
Montreal					
Short Period, Earthquake 1	2.01	9.37	0.291	0.05	1.02
Short Period, Earthquake 2	2.07	9.25	0.30	0.07	1.01
Long Period, Earthquake 1	2.41	10.8	0.348	0.13	1.18
Long Period, Earthquake 2	2.23	9.79	0.323	0.1	1.06
Vancouver					
Short Period, Earthquake 1	8.30	34.6	1.201	1.27	3.76
Short Period, Earthquake 2	5.36	23.8	0.776	0.70	2.58
Long Period, Earthquake 1	2.79	11.9	0.403	0.20	2.58
Long Period, Earthquake 2	2.41	10.9	0.348	0.13	1.24

Table 5.7: Ductility Demands, Bridge 2 (transverse direction)

Description	$\Delta_{max} * 10^{-3}$ (m)	$\theta_{max} * 10^{-3}$ (rad)	μ_{Δ}	μ_{ϕ}	μ_{θ}
Montreal					
Short Period, Earthquake 1	6.38	1.18	0.473	-	2.36
Short Period, Earthquake 2	5.42	1.01	0.404	-	2.02
Long Period, Earthquake 1	6.01	1.12	0.448	-	2.24
Long Period, Earthquake 2	6.25	1.17	0.466	-	2.33
Vancouver					
Short Period, Earthquake 1	19.4	3.30	1.445	6.19	6.60
Short Period, Earthquake 2	14.9	2.70	1.11	2.28	5.39
Long Period, Earthquake 1	9.20	1.72	0.686	-	3.43
Long Period, Earthquake 2	6.82	1.27	0.509	-	2.54

Table 5.8: Displacement Ductility Demand for Bridges with Underlying NC Soil Deposits

Depth (m)	City	Description	Bridge 1, X-dir		Bridge 1, Y-dir		Bridge 2	
			Δ_{max} $*10^{-2}$ (in)	μ_{Δ}	Δ_{max} $*10^{-2}$ (in)	μ_{Δ}	Δ_{max} $*10^{-3}$ (m)	μ_{Δ}
15	Montreal	Short P, E1	3.92	0.762	5.86	0.848	2.00	1.491
		Short P, E2	4.44	0.861	6.14	0.884	1.57	1.174
		Long P, E1	5.07	0.984	7.32	1.059	2.04	1.521
		Long P, E2	5.43	1.053	6.47	0.937	1.91	1.425
	Vancouver	Short P, E1	11.9	2.301	16.4	2.373	4.94	3.690
		Short P, E2	11.5	2.239	14.2	2.061	6.51	4.854
		Long P, E1	6.34	1.231	8.25	1.193	2.30	1.718
		Long P, E2	5.03	0.977	9.27	1.341	2.17	1.618
40	Montreal	Short P, E1	3.90	0.758	5.65	0.818	1.66	1.24
		Short P, E2	4.16	0.808	5.79	0.838	1.38	1.029
		Long P, E1	5.28	1.025	6.63	0.960	1.75	1.306
		Long P, E2	4.45	1.865	5.66	0.819	1.42	1.061
	Vancouver	Short P, E1	11.7	2.262	12.7	1.841	4.39	3.272
		Short P, E2	10.5	2.041	13.8	1.996	5.19	3.872
		Long P, E1	6.33	1.229	9.02	1.305	2.49	1.860
		Long P, E2	5.20	1.01	6.69	0.968	3.15	2.349

Table 5.9: Displacement Ductility Demand for Bridges with Underlying AS Soil Deposits

Depth (m)	City	Description	Bridge 1, X-dir		Bridge 1, Y-dir		Bridge 2	
			Δ_{max} $\cdot 10^{-2}$ (in)	μ_{Δ}	Δ_{max} $\cdot 10^{-2}$ (in)	μ_{Δ}	Δ_{max} $\cdot 10^{-3}$ (m)	μ_{Δ}
15	Montreal	Short P, E1	4.31	0.836	5.80	0.840	1.52	1.131
		Short P, E2	4.65	0.902	6.47	0.934	1.30	0.973
		Long P, E1	5.82	1.130	7.66	1.109	1.67	1.246
		Long P, E2	6.61	1.284	7.39	1.069	1.71	1.276
	Vancouver	Short P, E1	12.2	2.363	16.9	2.446	4.75	3.543
		Short P, E2	11.2	2.165	11.1	1.599	2.87	2.138
		Long P, E1	6.85	1.330	8.95	1.295	1.98	1.480
		Long P, E2	5.49	1.065	6.80	0.984	1.81	1.347
40	Montreal	Short P, E1	2.37	0.460	5.65	0.583	1.20	0.898
		Short P, E2	3.26	0.632	5.79	0.643	1.15	0.857
		Long P, E1	3.37	0.655	6.63	0.837	1.45	1.080
		Long P, E2	3.40	0.660	5.66	0.690	1.45	1.082
	Vancouver	Short P, E1	11.2	2.165	12.7	1.618	3.62	2.701
		Short P, E2	8.55	1.661	13.8	1.446	2.92	2.177
		Long P, E1	4.54	0.881	9.02	0.986	1.75	1.307
		Long P, E2	3.71	0.721	6.69	0.885	1.57	1.169

Table 5.10: Displacement Ductility Demand for Bridges with Underlying DS Soil Deposits

Depth (m)	City	Description	Bridge 1, X-dir		Bridge 1, Y-dir		Bridge 2	
			Δ_{max} $\ast 10^{-2}$ (in)	μ_{Δ}	Δ_{max} $\ast 10^{-2}$ (in)	μ_{Δ}	Δ_{max} $\ast 10^{-3}$ (m)	μ_{Δ}
15	Montreal	Short P, E1	4.09	0.793	3.32	0.480	1.22	0.912
		Short P, E2	5.00	0.970	3.73	0.540	1.47	1.096
		Long P, E1	3.61	0.701	5.07	0.734	1.34	0.996
		Long P, E2	4.82	0.936	4.68	0.677	1.44	1.076
	Vancouver	Short P, E1	11.4	2.216	11.1	1.599	4.14	3.090
		Short P, E2	9.17	1.780	12.2	1.761	3.43	2.556
		Long P, E1	4.68	0.908	4.66	0.674	2.55	1.903
		Long P, E2	5.03	0.976	4.00	0.579	1.41	1.055
40	Montreal	Short P, E1	4.27	0.829	5.73	0.829	1.50	1.116
		Short P, E2	4.46	0.865	6.50	0.941	1.26	0.942
		Long P, E1	5.75	1.117	7.36	1.065	1.63	1.215
		Long P, E2	6.49	1.259	7.26	1.051	1.67	1.249
	Vancouver	Short P, E1	12.1	2.342	16.6	2.402	5.36	3.998
		Short P, E2	11.1	2.151	11.5	1.664	2.78	2.074
		Long P, E1	6.63	1.287	8.90	1.287	1.86	1.386
		Long P, E2	5.31	1.031	6.78	0.981	1.75	1.309

Table 5.11: Displacement Ductility Demand (2%/50 years probability of exceedance)

City	Description	Bridge 1, X-dir		Bridge 1, Y-dir		Bridge 2	
		Δ_{max} *10 ⁻² (in)	μ_{Δ}	Δ_{max} *10 ⁻² (in)	μ_{Δ}	Δ_{max} *10 ⁻³ (m)	μ_{Δ}
Montreal	Short P, E1	3.12	0.606	4.12	0.596	1.31	1.248
	Short P, E2	4.85	0.942	4.93	0.714	1.98	1.885
	Short P, E3	3.23	0.627	4.17	0.604	1.51	1.437
	Short P, E4	3.12	0.606	6.19	0.894	1.66	1.581
	Long P, E1	4.34	0.843	7.05	1.021	2.79	2.655
	Long P, E2	4.21	0.817	5.56	0.804	1.69	1.613
	Long P, E3	3.71	0.720	5.40	0.781	2.30	2.187
	Long P, E4	4.34	0.843	6.08	0.880	1.99	1.895
Vancouver	Short P, E1	8.99	1.746	11.5	1.664	2.86	2.722
	Short P, E2	10.7	2.082	15.3	2.217	4.93	4.690
	Short P, E3	12.4	2.404	18.6	2.685	6.15	5.852
	Short P, E4	7.88	1.529	9.2	1.332	3.51	3.343
	Long P, E1	6.49	1.268	8.36	1.209	2.54	2.419
	Long P, E2	5.33	1.034	7.13	1.031	2.34	2.229
	Long P, E3	6.54	1.269	8.08	1.169	4.00	3.811
	Long P, E4	6.49	1.260	6.57	0.951	2.17	2.065

(a) Equal Max Displacement Response

(b) Equal Max Potential Energy

Figure 5.1: Assumed seismic response of a single-column Bridge bent

(a) At Yield

(b) At Subsequent Post Elastic Hinge Rotation

Figure 5.2: Deflected shape of a single-column bridge bent

(a) Elevation and Plan Views

(b) Abutment Details

(c) Force-Displacement Relationship of Energy-Absorbing Device

Figure 5.3: Two-span Bridge 1

(a) Joint Numbering

(b) Nonlinear Column Elements and Curvature Designation

(c) Nonlinear Hinge Element at Abutments

(d) Moment Single Curvature

Figure 5.4: Structural idealization - Bridge 1

Figure 5.5: Yield interaction surface description

(a) Bridge Elevation

(b) Typical Deck Cross Section

(c) Finite Element Model

Figure 5.6: Structural idealization - Bridge 2

Chapter 6

Conclusion

6.1 Summary and Conclusions

In the present study, the effects of using uniform hazard spectra on soil amplification behavior and ductility demand of bridges are investigated. The free field surface response spectra of four types of homogeneous soil in three depths due to the specified ground motions are calculated. The parameters included in soil amplification effects are: soil type, soil depth and frequency content of the seismic actions. The adequacies of design spectra presented in the five major bridge design codes, considered in the comparison study, are verified using the results obtained from soil amplification effects. The UHS compatible simulated ground motions with 10% and 2% probability of exceedance in 50 years, for Montreal in the east and Vancouver in the west, are considered in the analyses. The effects of using a higher probability level (2% probability of exceedance in 50 years) on soil amplifications and ductility demand of bridges are evaluated. The computer program FLUSH is employed to carry out the analyses on the soil deposits. Nonlinear analyses of the bridges are carried out using NEABS computer program. The main conclusions are presented as follows:

- The effect of soil amplification is less significant if natural frequency of the soil deposit does not coincide with the frequency range where the seismic energy of the ground motions is concentrated.
- The vibration period of the normal to lightly overconsolidated clay soil deposits

at peak spectra ratio, is the longest among the soil types considered in the study, whereas dense sand has the shortest one. The amplification effect of heavily overconsolidated clay and alluvial sand and silt soil deposits are similar.

- For the short period component of the motions (less than 0.1 s) the free field spectral ratio can be less than one and as low as 0.5, especially in the case of normal to lightly over consolidated clay. This implies that the structure in such period range may experience less intensive seismic ground motions than the same structure located on rock sites.
- The free field spectral ratios of all soil deposits converges and towards one for the long period component of the motions (longer than 8 s), which indicates that the structure in such period range will not be exposed to the amplification effects of soil deposits.
- The peak spectral acceleration and the peak free field spectral ratio of the soil deposits occur at two different vibrational periods for each of the different soil deposits. The periods when the peaks occur, is not effected by the severity of the seismic hazard considered in the analyses.
- The design spectra presented in the CALTRAN seismic provisions are the closest ones to the spectra of analytical results, compared to the spectra of the other bridge codes.
- The seismic characteristics are different in eastern and western regions of Canada. Because of differences in seismic characteristics of eastern and western Canada, the response spectra developed for these two regions are also different. Therefore using the same design spectra in the seismic provisions of the code for all the regions across the country will not be appropriate.
- In Montreal, the average displacement ductility demand caused by long period events is larger than that from the short period ones. This is in contrast with the

results obtained in Vancouver, where the larger displacement ductility demands belong to short period earthquakes.

6.2 Recommendations for Future Work

- The analysis and, therefore, the resulting conclusions are made only for two locations in Canada, Montreal and Vancouver. To cover all the locations in Canada and obtain more statistical data, more analyses should be performed using the specific hazard values of each location. In particular, locations where earthquakes with low a/v ratio frequency content should be studied.
- The soil deposits considered in the present study consist of one homogeneous soil profile. The soil amplification effects of the sites with more complex geometric characteristics of the local soils are required before any soil factor can be introduced to be used in the seismic provisions of bridge design code.
- More studies are needed in order to investigate the behavior of multi-span bridges with structural properties different from the cases considered in the present study, under different seismic actions. More experimental and analytical work is also required to obtain information about the characteristics and location of the plastic hinges in the different components of the bridge substructure. The ductility demand of other bridge components such as connections should also be studied.

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