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EVALUATION OF USING UNIFORM HAZARD SPECTRA FOR SEISMIC DESIGN OF CANADIAN HIGHWAY BRIDGES

By

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the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements for the degree of

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Abstract

In recent years the method of defining seismic hazard and performing seismic analysis has advanced. As a result the 2005 edition of the National Building Code of Canada has made numerous changes to its seismic design methodology. In 2003 the ATC published seismic design guidelines for use with the AASHTO bridge design specification, which incorporate similar changes into their seismic design methodology. The impact and significance of using the most recent seismic hazard definition proposed by the Geological Survey of Canada and NEHRP site factors on the design and performance of two typical highway bridges located in four Canadian cities are studied. The displacement ductility demand and rotational ductility demand are used to evaluate the performance of the bridges using ground motions compatible with the 10% in 50 year UHS and 2% in 50 year UHS. Seismic design of Canadian Highway bridges using more delineated performance objectives, the 2% in 50 year UHS and NEHRP site factors are evaluated.

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Table of Contents

Abstract.....	iii
Acknowledgements.....	iv
Table of Contents.....	v
List Of Tables	vii
List Of Figures.....	ix
List of Abbreviations, Notations and Symbols.....	xi
1. Introduction.....	1
1.1 General.....	1
1.1.1 Uniform Hazard Spectra	2
1.1.2 Probability of Exceedance Level	3
1.2 Fourth Generation Seismic Hazard Maps of Canada.....	4
1.3 2005 National Building Code of Canada.....	5
1.4 Recommended LRFD Guidelines for Seismic Design of Highway Bridges.....	5
1.5 Objectives and Scope of the Thesis	6
2. Overview of Code Provisions for Seismic Design	9
2.1 Introduction.....	9
2.2 National Building Code of Canada.....	9
2.2.1 Definition of Seismic Hazard.....	9
2.2.2 Site Factor	10
2.2.3 Importance Factor	11
2.2.4 Higher Mode Effects and Base Overturning.....	12
2.2.5 Ductility and Overstrength Factors	12
2.2.6 Dynamic Analysis Requirements.....	13
2.3 AASHTO LRFD Bridge Specifications and ATC/MCEER Guidelines.....	13
2.3.1 Definition of Seismic Hazard.....	14
2.3.2 Design Earthquake	15
2.3.3 Spectral Shapes	15
2.3.4 Soil Factor.....	16
2.3.5 Importance Classification and Performance Objectives	16
2.3.6 Analysis and Design Methods	17
2.3.7 Design Incentives.....	20
2.4 Canadian Highway Bridge Design Code	20
2.4.1 Definition of Seismic Hazard.....	20
2.4.2 Design Earthquake and Performance Objectives.....	21
2.4.3 Spectral Shapes	21
2.4.4 Site Effects	21
2.4.5 Importance Classification	21
2.4.6 Analysis and Design Methods	22
3. Seismic Hazard Parameter and Spectral Shape	33
3.1 1995 National Building Code of Canada	33
3.2 2005 National Building Code of Canada	33
3.3 Comparison of the Spectral Shape of 1995 NBCC and 2005 NBCC	35
3.4 Canadian Highway Bridge Design Code and AASHTO	35
3.5 2003 ATC/MCEER.....	36

3.6	Comparison of Spectral Shape of CHBDC, ATC and UHS	37
3.7	Development of UHS Compatible Ground Motions	38
3.8	Summary	40
4.	Displacement Ductility and Force Reduction Factors	73
4.1	General	73
4.2	Displacement Ductility and Force Reduction Factors	74
4.3	CHBDC/AASHTO Force Reduction Factors	76
4.4	2003 ATC/MCEER Response Modification Factors.....	76
4.5	Development of Displacement Ductility Spectra	76
4.5.1	Constant Strength-Inertia Ratio Displacement Ductility Spectra	78
4.5.2	Constant Strength Displacement Ductility Spectra.....	80
4.6	Summary	82
4.7	Conclusions.....	84
5.	Characteristics of Bridge Structures	94
5.1	General.....	94
5.2	Concrete Stress-Strain Relationship	95
5.3	Column Plastic Hinge Properties.....	96
5.4	Barnsdale Road Underpass	97
5.5	CNR Overhead EBL	98
6.	Dynamic Time History Analysis	116
6.1	General.....	116
6.2	Analysis Techniques	117
6.2.1	Yield Displacement and Rotation	118
6.2.2	Recommended Response Modification Factors.....	119
6.3	Dynamic Analysis.....	120
6.3.1	Barnsdale Road Underpass	120
6.3.2	CNR Overhead EBL	122
6.4	Dynamic Analysis using Site Class A to E.....	123
6.5	Design Displacement and Rotation.....	125
6.5.1	Plastic Hinge Length.....	126
6.5.2	Minimum Seat Width.....	127
6.5.3	Plastic Rotation Capacities	128
6.6	Summary	129
7.	Summary and Conclusion.....	168
7.1	Summary and Conclusion.....	168
7.2	Recommendations for Future Study	171
	REFERENCES	172

List Of Tables

Table 2.1 Overview of Seismic Provisions used in the 1995 and 2005 NBCC.....	23
Table 2.1 (Continued) Overview of Seismic Provisions used in the 1995 and 2005 NBCC	24
Table 2.2 Soil Classification and Site Coefficient 1995 NBCC and 2000 CHBDC/ AASHTO	25
Table 2.3 2005 NBCC Soil Profile / 2003 MCEER/ATC Site Classification (Based on NEHRP recommendations).....	26
Table 2.4 NBCC Fa Values	26
Table 2.5 NBCC Fv Values	26
Table 2.6 2005 NBCC Higher Mode Factor and Base Overturning factor	27
Table 2.7 Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications	28
Table 2.7 (Continued) Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications.....	29
Table 2.7 (Continued) Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications.....	30
Table 2.8 2003 ATC Fa Values	30
Table 2.9 2003 ATC Fv Values	30
Table 2.10 AASHTO Seismic Performance Category	31
Table 2.11 ATC/MCEER 2003 Guidelines for the Seismic Design of Highway Bridges Design Earthquakes and Seismic Performance Objectives.....	31
Table 2.12 ATC/MCEER 2003 Guidelines for the Seismic Design of Highway Bridges Seismic Hazard Levels-Based on MCE	31
Table 2.13 2003 ATC/MCEER Seismic Design Requirements (SDR).....	32
Table 2.14 2000 CHBDC Performance Requirements	32
Table 2.15 2000 CHBDC Seismic Performance Zones.....	32
Table 2.16 2000 CHBDC Minimum Analysis Requirements for Multispan Bridges	32
Table 3.1 1995 NBCC Seismic Design Parameters.....	42
Table 3.2 1995 NBCC Seismic Response Factor	42
Table 3.3 UHS Values (Robust method and 50% confidence level).....	42
Table 3.4 Ground Motion records matching 10% in 50 year UHS	43
Table 3.5 Ground Motion records matching 2% in 50 year UHS	43
Table 3.6 Relative value of 2% in 50 year and 10% in 50 year UHS spectral acceleration in the short(<0.5s), intermediate (0.5-1.0s) and long period ranges (>1.0s) with respect to the previous code values.....	44
Table 4.1 CHBDC/AASHTO Response Modification Factor.....	85
Table 4.2 ATC/MCEER Response Modification Factors.....	86
Table 5.1 Barnsdale Road Underpass Reinforced Concrete Properties.....	101
Table 5.2 CNR Overhead EBL Reinforced Concrete Properties.....	101
Table 5.3 Barnsdale Road Overpass	101
Table 5.4 CNR Overhead EBL	101
Table 6.1 Barnsdale Road Overpass Frequencies.....	131
Table 6.2 Barnsdale Road Overpass Modal Mass Participation.....	131
Table 6.3 CNR Overhead EBL	132

Table 6.4 CNR Overhead EBL Modal Mass Participation.....	132
Table 6.5 Yield Displacement values used in Ductility Analysis.....	133
Table 6.6 Percent Difference of the maximum displacement response of the NONSPEC program w.r.t. the NEABS program in the longitudinal direction.....	133
Table 6.7 Comparison of Barnsdale Road Overpass Column Displacement and Displacement Ductility, Longitudinal Direction.....	134
Table 6.8 Column Maximum Displacement and Displacement Ductility	134
Table 6.9 Column Maximum Rotation and Rotational Ductility	135
Table 6.10 2% in 50 year Ground Motions matching Soil Types A-E.....	136
Table 6.11 2% in 50 year Ground Motions matching Soil Types A-E.....	136
Table 6.12 2% in 50 year Ground Motions matching Soil Types A-E.....	136
Table 6.13 Barnsdale Road Underpass Displacement Soil Types A-E	137
Table 6.14 Barnsdale Road Underpass Rotation Soil Types A-E	137
Table 6.15 Barnsdale Road Underpass Seat Width and Rotation Capacity	138
Table 6.16 CNR Overhead EBL Seat Width and Rotation Capacity	138

List Of Figures

Figure 3.1 1995 NBCC Seismic Response Factor	45
Figure 3.2 Spectral Shapes 1995 NBCC.....	46
Figure 3.3 Montreal/Ottawa Spectral Shapes	47
Figure 3.4 Toronto Spectral Shapes.....	48
Figure 3.5 Vancouver Spectral Shapes	49
Figure 3.6 Ottawa 2000 CHBDC, 2002 AASHTO Method	50
Figure 3.7 AASHTO/CHBDC Spectra Soil Type 1	51
Figure 3.8 Ottawa 2003 ATC Method	52
Figure 3.9 2003 ATC Method, Site Class C	53
Figure 3.10 Ottawa Spectral Shapes	54
Figure 3.11 Montreal Spectral Shapes	55
Figure 3.12 Toronto Spectral Shapes.....	56
Figure 3.13 Vancouver Spectral Shapes	57
Figure 3.14 Montreal/Ottawa UHS Matched Time History Records	58
Figure 3.15 Toronto UHS Matched Time History Records.....	59
Figure 3.16 Vancouver UHS Matched Time History Records	60
Figure 3.17 Montreal/Ottawa 10% In 50 Year Short Period Time History Record	61
Figure 3.18 Montreal/Ottawa 10% In 50 Year Long Period Time History Record	62
Figure 3.19 Montreal/Ottawa 2% In 50 Year Short Period Time History Record	63
Figure 3.20 Montreal/Ottawa 2% In 50 Year Long Period Time History Record	64
Figure 3.21 Toronto 10% In 50 Year Short Period Time History Record.....	65
Figure 3.22 Toronto 10% In 50 Year Long Period Time History Record.....	66
Figure 3.23 Toronto 2% In 50 Year Short Period Time History Record.....	67
Figure 3.24 Toronto 2% In 50 Year Long Period Time History Record	68
Figure 3.25 Vancouver 10% In 50 Year Short Period Time History Record	69
Figure 3.26 Vancouver 10% In 50 Year Long Period Time History Record	70
Figure 3.27 Vancouver 2% In 50 Year Short Period Time History Record	71
Figure 3.28 Vancouver 2% In 50 Year Long Period Time History Record	72
Figure 4.1 Positive Displacement Ductility 10% In 50 Year Short Period Ground Motion	87
Figure 4.2 Positive Displacement Ductility 10% In 50 Year Long period Ground Motion	88
Figure 4.3 Positive Displacement Ductility 2% In 50 Year Short Period Ground Motion	89
Figure 4.4 Positive Displacement Ductility 2% In 50 Year Long Period Ground Motion.....	90
Figure 4.5 Montreal/Ottawa Positive Displacement Ductility.....	91
Figure 4.6 Toronto Positive Displacement Ductility	92
Figure 4.7 Vancouver Positive Displacement Ductility	93
Figure 5.1 Kent And Park Stress-Strain Relationship	102
Figure 5.2 Mander, Priestly And Park Stress-Strain Relationship.....	103
Figure 5.3 Biax Modified Kent And Park Stress-Strain Relationship.....	104
Figure 5.4 Column Displacement Single Curvature	105
Figure 5.5 Column Displacement Double Curvature	106
Figure 5.6 Elevation View Barnsdale Road Overpass.....	107

Figure 5.7 Elevation And Cross Section View Barnsdale Road Overpass Column.....	108
Figure 5.8 Cross Section View Barnsdale Road Overpass Deck.....	109
Figure 5.9 Barnsdale Road Underpass Interaction Diagram	110
Figure 5.10 Moment Curvature-Barnsdale Rd. Overpass.....	111
Figure 5.11 Elevation View CNR Overhead EBL.....	112
Figure 5.12 Elevation And Cross Section View CNR Overhead Column	113
Figure 5.13 Interaction Diagram-CNR Overpass	114
Figure 5.14 Moment Curvature-CNR Overpass	115
Figure 6.1 Barnsdale Road Underpass Node Placement	139
Figure 6.2 CNR Overhead EBL Node Placement	139
Figure 6.3 CNR interior column bent	139
Figure 6.4 NONSPEC Single Degree of Freedom Model	140
Figure 6.5 Montreal/Ottawa 10% In 50 Year Short Period Displacement Time History	141
Figure 6.6 Montreal/Ottawa 10% In 50 Year Long Period Displacement Time History	142
Figure 6.7 Montreal/Ottawa 2% In 50 Year Short Period Displacement Time History.	143
Figure 6.8 Montreal/Ottawa 2% In 50 Year Long Period Displacement Time History.	144
Figure 6.9 Toronto 10% In 50 Year Short Period Displacement Time History	145
Figure 6.10 Toronto 10% In 50 Year Long Period Displacement Time History	146
Figure 6.11 Toronto 2% In 50 Year Short Period Displacement Time History	147
Figure 6.12 Toronto 2% In 50 Year Long Period Displacement Time History	148
Figure 6.13 Vancouver 10% In 50 Year Short Period Displacement Time History.....	149
Figure 6.14 Vancouver 10% In 50 Year Long Period Displacement Time History.....	150
Figure 6.15 Vancouver 2% In 50 Year Short Period Displacement Time History.....	151
Figure 6.16 Vancouver 2% In 50 Year Long Period Displacement Time History.....	152
Figure 6.17 CNR Montreal/Ottawa 10% In 50 Year Short Period Displacement Time History.....	153
Figure 6.18 CNR Montreal/Ottawa 10% In 50 Year Long Period Displacement Time History.....	154
Figure 6.19 CNR Montreal/Ottawa 2% In 50 Year Short Period Displacement Time History.....	155
Figure 6.20 CNR Montreal/Ottawa 2% In 50 Year Long Period Displacement Time History.....	156
Figure 6.21 CNR Toronto 10% In 50 Year Short Period Displacement Time History..	157
Figure 6.22 CNR Toronto 10% In 50 Year Long Period Displacement Time History ..	158
Figure 6.23 CNR Toronto 2% In 50 Year Short Period Displacement Time History	159
Figure 6.24 CNR Toronto 2% In 50 Year Long Period Displacement Time History	160
Figure 6.25 CNR Vancouver 10% In 50 Year Short Period Displacement Time History	161
Figure 6.26 CNR Vancouver 10% In 50 Year Long Period Displacement Time History	162
Figure 6.27 CNR Vancouver 2% In 50 Year Short Period Displacement Time History	163
Figure 6.28 CNR Vancouver 2% In 50 Year Long Period Displacement Time History	164
Figure 6.29 Montreal/Ottawa 2% In 50 Year UHS Soil Type A-E	165
Figure 6.30 Toronto 2% In 50 Year UHS Soil Type A-E	166
Figure 6.31 Vancouver 2% In 50 Year UHS Soil Type A-E.....	167

List of Abbreviations, Notations and Symbols

A	Acceleration Coefficient
AASHTO	American Association of State and Highway Transportation Officials
ATC	Applied Technology Council
BIAX	Computer program evaluates uniaxial and biaxial strength of reinforced concrete, developed by Wallace, Clarkson University, New York.
C	Damping Ratio
CALTRANS	California Department of Transportation
CHBDC	Canadian Highway Bridge Design Code
CNR	Canadian National Railway
C_s	Elastic Seismic Response Coefficient
C_{sm}	Elastic Seismic Response Coefficient
EE	Expected Earthquake
F	Foundation Factor
F_a	Short period Soil Factor
F_v	Long period Soil Factor
g	Force of Gravity
GSC	Geological Survey of Canada
I	Importance Factor
IC	Importance Category
I_e	Importance factor for earthquakes
J_v	Moment Reduction Factor
K	Stiffness
L	Length
L_p	Plastic Hinge length
LRFD	Load and Resistance Factor Design
M	Mass
MCE	Maximum Credible Earthquake
MCEER	Multi-disciplinary Center for Earthquake Engineering Research
M_v	Higher Mode Factor
NBCC	National Building Code of Canada
NEABS	Nonlinear earthquake analysis of bridge systems computer program developed by Penzien, Imbsen and Liu, University of California at Berkeley
NEHRP	National Earthquake Hazards Reduction Program
NONSPEC	Determines inelastic response spectra to dynamic excitation, computer program developed by Mahin and Lin, University of California at Berkeley
P_e	Uniformly distributed static seismic loading
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
P_o	Uniform arbitrary lateral load
R	Force Reduction Factor
R_b	Base Response Modification Factor
R_d	Ductility Reduction Factor

R_o	Overstrength Reduction Factor
R_y	Yield force
S	Seismic Response Factor
S	Site Coefficient
S_a	Spectral Acceleration
S_s	0.2s spectral acceleration
S_l	1.0s spectral acceleration
SDAP	Seismic Design and Analysis Procedure
SDR	Seismic Design Requirements
SPC	Seismic Performance Category
SPT	Standard Penetration Test
T	Period
USGS	United States Geological Survey
UHS	Uniform Hazard Spectra
v	Zonal Velocity Ratio
V_s	Shear Wave Velocity
W	Weight
Z_a	Acceleration Related Seismic Zone
Z_v	Velocity Related Seismic Zone
Δ	Displacement
Δ_y	Yield Displacement
ϵ_{50u}	Concrete Strain at 50% ultimate unconfined strength
ϵ_{50c}	Concrete Strain at 50% ultimate confined strength
ϵ_{50h}	Concrete Strain at 50% hoop strength
ϵ_{cc}	Confined Concrete Strain at peak stress
E_c	Modulus of Elasticity of concrete
ϵ_o	Unconfined Concrete Strain at peak stress
f'_c	Compressive Strength of unconfined concrete
f'_t	Concrete tensile strength
F'_{cc}	Compressive Strength of confined concrete
ϕ	Curvature
ϕ_y	Yield Curvature
θ	Rotation
θ_y	Yield Rotation
θ_p	Plastic Rotation
μ	Ductility
μ_ϕ	Curvature Ductility
μ_θ	Rotational Ductility
μ_Δ	Displacement Ductility
$\ddot{u}(t)$	Acceleration as a function of time
$\dot{u}(t)$	Velocity as a function of time
$u(t)$	Displacement as a function of time

u_y	Yield Displacement
$\ddot{u}_g$	Acceleration of the ground
ω	Angular Frequency
η	Strength-Inertia Force Ratio
ν	Poisson's Ratio
γ	Unit Weight
ξ	Damping Ratio

Chapter 1

1. Introduction

1.1 General

The design of earthquake resistant structures has become increasingly important for engineers over the past 30 years. Numerous advances and developments in the fields of Structural Engineering, Seismology, Material Science, Computer analysis methods and Geology have led to continual improvements in design methods and earthquake engineering practices.

Highway bridges are important structures in Canada, as in other parts of the world, for they are vital to the economic well being of society, public safety and national defense. The bridge structures in the national transportation system must be able to perform adequately when subjected to earthquake loadings, while being as cost effective as possible.

In Canada, seismic hazard levels specified for seismic design of structures in national design codes are developed by the Geological Survey of Canada (GSC). The GSC has recently made significant changes to the way seismic hazards are quantified. These new specifications of seismic hazard are currently being incorporated in earthquake engineering practices in Canada. The GSC developed the Fourth generation of seismic hazard maps of Canada, which form the basis for the seismic design provisions in the new edition of the National Building Code of Canada (NBCC) published in 2005.

The Canadian Highway Bridge Design Code (CHBDC) is the national design standard for bridge structures in Canada. CHBDC 2000 was first published in 2000 and adopted the seismic hazard provisions of NBCC 1995. Because of the recent changes in

the specifications of seismic hazard by GSC and their incorporation in NBCC 2005, the impact and significance of the new seismic hazard specifications to the seismic design provisions in CHBDC 2000 need to be assessed.

1.1.1 Uniform Hazard Spectra

The fourth generation seismic hazard maps of Canada created by GSC are derived using the uniform hazard spectrum concept. Conceptually, a uniform hazard spectrum (UHS) represents the seismic hazard at the same probability level at a site location over the entire vibration period range. The uniform hazard spectrum is the envelope function of earthquake hazard resulting from all earthquakes contributing to the seismic hazard at the site. It is the composite result of many different events of varying magnitude and distance. The maximum response of a single degree of freedom system to all earthquake events of the same specified probability of exceedance, which is taken as 2% in 50 years for the fourth generation seismic hazard maps by GSC, are plotted as an envelope function giving the uniform hazard spectrum of the site. Considering this, the UHS values at specific period values are contributed to from various earthquake events of different magnitude and distance. For example, the earthquake hazard at a period of 0.6 seconds is probably the result of a different earthquake from the event that gives the hazard at a period of 1.6 seconds, however the probability of exceeding the spectral coordinates for the two earthquakes is the same.

Previously the shape of the earthquake hazard spectrum had been developed based on the ground motion relationships for California by scaling the spectrum using the PGA value to the specified level for a given Canadian location. The scaled spectrum approach means that only the PGA value actually corresponds to the design probability

level. In the development of the fourth generation of seismic maps by GSC, new ground motion relationships for eastern and western Canada are adopted which more accurately reflect the unique magnitude and distance relationships of those regions in Canada. The shape of the UHS is more representative of the actual spectra at a given location in Canada compared to the previously used scaled spectrum approach.

1.1.2 Probability of Exceedance Level

Traditionally seismic hazard in Canada and the United States is defined at a probability of exceedance level of 10% in 50 years (475 year return period). Studies and experience have shown that due to conservatism inherent in design codes and engineering practices, the probability of structural collapse is considerably lower than 10% in 50 years. The amount of conservatism varies from eastern to western Canada because of differences in the shape and slope of the hazard curve, meaning that while a structure in eastern Canada may have a 1 in 1500 probability of collapse, the same structure in western Canada may have a 1 in 2500 probability of collapse (Adams et al., 2003).

In order to achieve uniform reliability in seismic design, studies have shown that design amplitudes, which correspond to the target reliability level, should be used. In Canada the target reliability for structures against collapse due to seismic forces is 2% in 50 years (1 in 2500). Therefore in order to achieve a uniform probability of 1 in 2500 for structural collapse against earthquake forces, the earthquake ground motion used in design should correspond to a 1 in 2500 year return period, or 2% in 50 year probability of exceedance (Heidebrecht, 2003). The seismic hazard maps developed by GSC, for incorporation in NBCC 2005 have a 2% in 50 year probability of exceedance level.

The change from 10% in 50 years to 2% in 50 years reflects similar changes adopted by the United States Geological Survey (USGS) in the United States. The USGS has updated the seismic hazard in the United States, and included a probability level of 2% in 50 years for use in design.

1.2 Fourth Generation Seismic Hazard Maps of Canada

The purpose of updating the seismic hazard maps of Canada is to provide more rational, uniform and safe earthquake engineering design throughout Canada. Recent research in Canada and around the world has led to a better understanding of seismic activities and their estimation in Canada. These advances and the resulting insights and data have been incorporated in the development of the new seismic hazard maps. The major changes in the evaluation of seismic hazard by GSC in developing the fourth generation seismic hazard maps are as follows (Adams et al., 2003):

- The seismic hazard is described in terms of the uniform hazard spectra.
- The probability of exceedance level of the seismic hazard is specified at 2% in 50 years (approximately 2500 year return period) instead of 10% in 50 years (approximately 475 year return period).
- The elimination of the designation of seismic zones. Seismic hazard is now specified by uniform hazard spectra, specific for each location.
- Improved source models are used in developing the seismic models for eastern and western Canada.
- Uncertainty in the hazard is considered by using the median spectral values.
- The reference ground condition, defined as site class C, is taken as soil deposits with shear wave velocity between 360-750m/s.

1.3 2005 National Building Code of Canada

Throughout the history of NBCC, seismic design provisions have continually evolved following the advances of research in engineering and seismology. The most recent changes to the seismic design provisions are the result of advances in seismology and geology, earthquake engineering research, increased understanding of structural performance during earthquakes and advances in the development of seismic design codes around the world. In addition to changes in the definition of seismic hazard in the 2005 NBCC, there are other significant changes in design, such as:

- use of period dependent site factors
- more detailed definition of overstrength and ductility factors
- improved Method to account for higher mode effects
- acceptance of Dynamic Analysis as the primary method of design

1.4 Recommended LRFD Guidelines for Seismic Design of Highway Bridges

In the United States, the American Association of State Highway Transportation Officials (AASHTO, 1998) publishes guidelines for bridge design. As in Canada, earthquake design specifications in the United States have undergone significant changes over the past 30 years. The United States Geological Survey (USGS) updated the national earthquake hazard maps in the United States in 1996 (Frankel et al., 1996). Changes in the latest seismic hazard maps by the USGS include changes in the probability of exceedance level and the presentation of seismic hazard by spectral coordinates. To assess the significance of these changes in the definition of seismic hazard and other advances in earthquake engineering research, ATC/MCEER has recently published a guideline report (ATC/MCEER, 2003) on seismic design of bridges,

which considers the possible incorporation of the new seismic hazard information and its new format in the future edition of AASHTO LRFD Bridge Design Specifications (AASHTO, 1998). Major changes proposed in the guidelines to the existing design code provisions include:

- presentation of seismic hazard in spectral format
- use of three different probability of exceedance levels, 10% in 50 years (approximately 475 year return period), 5% in 50 years and 2% in 50 years (approximately 2500 year return period)
- more detailed specifications of performance objectives
- design incentives for performing more sophisticated types of analysis, such as ‘push-over analysis’
- use of period dependent site factors
- new spectral shapes
- enhanced liquefaction design requirements

1.5 Objectives and Scope of the Thesis

The National Building Code of Canada has updated its seismic provisions for structural design in Canada including the adoption of seismic hazard specified by uniform hazard spectra. The changes provide a more accurate method of delineating the effects of the overlying soil on the seismic forces imposed on a structure and encourage the use of more sophisticated analysis and design methods. The current edition of the Canadian Highway Bridge design code, CHBDC 2000, has seismic hazard specifications based on seismic hazard map information published by GSC in 1985, which is now obsolete. The analysis and design methods used in CHBDC 2000 are essentially the same as those

published by AASHTO 20 years ago. Therefore, there is the need to evaluate the impact and significance of recent developments and adoption of the UHS format of seismic hazard specification in the 2005 NBCC on the bridge design code practices in Canada.

The objective of this thesis is to obtain insights in light of the recent developments in earthquake engineering design that will lead to better performance objectives and use the latest scientific and engineering knowledge to perform safer and more efficient seismic design of highway bridges in Canada.

The seismic design code methods of NBCC, AASHTO and CHBDC are studied and compared. Displacement Ductility spectra for typical bridges located in Montreal, Ottawa, Toronto, and Vancouver are evaluated based on acceleration time histories compatible with the UHS of probability 10% and 2% in 50 years as provided by GSC. The ductility of typical bridge structures common in Canada are evaluated by non-linear time history analysis, using ground motions compatible with the 10% and 2% in 50 years UHS. The ductility found using non-linear time history analysis and the recommended force reduction factors in CHBDC 2000 and AASHTO code procedures are compared. Ground motions compatible with the short and long period UHS are used to compare the effect of the 2005 NBCC soil factors on the seismic response of Barnsdale Road Underpass.

The thesis is divided into seven Chapters:

- 1) Introduction: The various design codes are discussed and the objectives and scope of the thesis are presented.

- 2) Overview of Code Provisions for seismic design: The methods of seismic design in The National Building Code of Canada, AASHTO and Canadian Highway Bridge design are discussed.
- 3) Seismic Hazard Parameters and Spectral Shape: The spectral shape in NBCC 1995 is compared to the shape of the UHS. The spectral shape of CHBDC 2000 and AASHTO bridge design codes are compared to the corresponding recommendations in the 2003 ATC/MCEER guideline and the UHS. The development of ground motion time histories compatible with the 10% in 50 year and 2% in 50 year UHS are presented.
- 4) Displacement Ductility Spectra: The principles of ductile design are discussed. The force reduction factors used in the various design codes are presented. Displacement Ductility spectra for Montreal/Ottawa, Toronto and Vancouver developed by the NONSPEC computer program are presented.
- 5) Ductile Design: The material properties and stress-strain relationships of the structural models used in the dynamic response analysis carried out in this study are discussed.
- 6) Dynamic Time History Analysis: The analyses and the results of two typical Canadian highway bridge structures are presented. The analysis results are compared to displacement limits recommended in ATC/MCEER guidelines.
- 7) Conclusion: A summary and conclusion is provided.

Chapter 2

2. Overview of Code Provisions for Seismic Design

2.1 Introduction

The seismic analysis and design procedures of NBCC, AASTO LFRD bridge design specifications and CHBDC 2000 are discussed. Recent changes in the building code of Canada and the American bridge design code are discussed. A summary of the seismic design methods is provided. Important values used in seismic design are tabulated.

2.2 National Building Code of Canada

The current edition of the National Building Code of Canada was published in 1995 and the seismic design method is based on seismic hazard maps produced by GSC in 1985. Since that time, GSC has developed new seismic hazard maps for Canada (Adams et al., 2003), which have been adopted in the 2005 edition of NBCC. Other updates to the seismic provisions of NBCC include improved site factors, modified ductility factor, material and structural system specific overstrength factor, and more stringent dynamic analysis requirements. A summary of the 1995 and 2005 NBCC seismic provisions is presented in Table 2.1.

2.2.1 Definition of Seismic Hazard

Seismic hazard in Canada is defined in NBCC 1995 by both peak ground velocity (PGV) and peak ground acceleration (PGA). These values were calculated for a probability of exceedance of 10% in 50 years, or a return period of approximately 475 years. The seismic hazards were presented either graphically as contour maps of PGA and PGV or in tabular form, for selected Canadian cities. Based on the PGA or PGV value, a locality was assigned an acceleration-related and velocity-related seismic zone

and the zonal velocity ratio (v) was determined, which was used directly in seismic design. Standard spectral shapes were scaled by the zonal velocity ratio. In recent years, GSC has produced the new fourth generation seismic hazard maps of Canada, which have been incorporated in the 2005 edition of NBCC. Seismic hazard is now defined by uniform hazard spectra at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s. The probability of exceedance of the seismic hazard specified now by means of the UHS has changed from 10% in 50 years to 2% in 50 years. The method of defining the seismic hazard and spectral shapes are discussed in Chapter 3.

2.2.2 Site Factor

The characteristics and behaviour of soil underlying a structure has a direct affect on the magnitude of ground motion the structure experiences. The foundation factor in NBCC 1995 increases the intensity of ground motion depending on the type of soil and ground conditions of the site. The foundation factor, F , is divided into four categories based on qualitative descriptions of the type of soil present. The foundation factor ranges from 1.0 to 2.0. The soil categories in NBCC 1995 are presented in Table 2.2. The foundation factor in NBCC 1995 reflects the fact that soft soil amplifies earthquake ground motions more than stiff soil. However it is not simply the soil type, which is responsible for amplifying ground motion, the magnitude of the input ground motion also has an influence on the level of amplification by the soil. Soil responds non-linearly to large input ground motions and the implication is that stronger ground motions are less amplified than weaker ground motions. When exposed to strong ground motion, the non-linear response of the soil effectively limits the amount of amplification. The increased level of ground shaking experienced by a structure also depends on the predominant

period of the structure. In the 2005 edition of NBCC, the 1994 National Earthquake Hazard Reduction Program (NEHRP) site categories and response factor are adapted to the reference ground condition for Canada. The site categories specified in 2005 NBCC are presented in Table 2.3. The reference ground condition adopted by 2005 NBCC is Site Class C, which is similar to the reference ground condition in 1995 NBCC. In NEHRP, the reference ground condition is site class B. The 1994 NEHRP method defines the site class by the shear wave velocity in the top 30 meters of soil. There are 6 site classes from A to F, decreasing in soil strength from A, hard rock to E, soft soil, with site class F, to denote particularly vulnerable soils. The site class is determined to obtain soil factors, F_a and F_v used in design. The value of F_a for a particular site is determined using the spectral acceleration coordinate at 0.2 seconds. The value of F_v for a particular site is determined using the spectral acceleration coordinate at 1.0 second. The values of F_a and F_v decrease with larger UHS spectral values at $S_a(0.2)$ and $S_a(1.0)$, because of non-linear soil behaviour and its energy absorption characteristics at stronger ground shaking. The factors of F_a and F_v in 2005 NBCC are presented in Table 2.4 and Table 2.5, respectively. The spectral coordinates are multiplied by the factor F_a for the period range of 0.5 seconds or less and F_v for the period range greater than 0.5 seconds. The values of F_a and F_v are similar, except for site class E, for which F_v at higher spectral acceleration levels is larger.

2.2.3 Importance Factor

Both 1995 and 2005 editions of NBCC use an importance factor I to categorize structures. NBCC 1995 assigned a value of $I=1.5$ to post-disaster buildings, $I=1.3$ to schools and $I=1.0$ to other buildings. Post-disaster buildings are those that are required to

be functional immediately following an earthquake. The 2005 NBCC uses an importance factor I_e for the ultimate limit state design. The values of I_e are similar to those used in NBCC 1995. Normal structures are assigned an $I_e = 1.0$, high importance category structures are assigned $I_e = 1.3$ and Post-Disaster structures are assigned an $I_e = 1.5$. A new category for low importance category buildings, such as farm buildings and minor storage buildings are assigned an $I_e = 0.8$.

2.2.4 Higher Mode Effects and Base Overturning

The 2005 NBCC adopts a higher mode factor (M_v) which increases the lateral earthquake force depending on the ratio of $S_a(0.2)/S_a(2.0)$, the type of lateral force resisting system and the fundamental period of the structure. The determination of a base overturning moment reduction factor (J_v) is modified from previous 1995 NBCC by making it dependant on the ratio of $S_a(0.2)/S_a(2.0)$, the type of lateral force resisting system and the fundamental period of the structure. The values of M_v and J_v are presented in Table 2.6.

2.2.5 Ductility and Overstrength Factors

In NBCC 1995, an R factor is used to reduce seismic forces based on the type of seismic force resisting system and the building material. For structures designed in accordance to CSA requirements, the R factor generally represents the amount of dissipated seismic energy or ductility the system is expected to exhibit or have. In NBCC 1995, the seismic force is multiplied by another factor, U, where $U=0.6$, to further reduce the design force. Structures designed using this method are expected to experience plastic deformation during the design earthquake without failure. For the 2005 edition of NBCC, a ductility reduction factor R_d is introduced. The ductility reduction factor

replaces the R factor in 1995 NBCC. The R_d factor is similar to the R factor in most cases. In 2005 NBCC the number of seismic force resisting systems (SFRS) specifically recognized in the design process has increased and the U factor has been replaced by the material overstrength reduction factor, R_o . The R_o factor is material specific and it represents the effects of overstrength resulting from member size, nominal material strength, actual yield strength, the effect of strain hardening and the formation of a failure mechanism within the structure. Table 2.1 compares the R factor used in 1995 NBCC and the R_d and R_o factors in 2005 NBCC. In the 2005 NBCC the height of the structure is limited if the spectral acceleration exceeds a certain threshold.

2.2.6 Dynamic Analysis Requirements

The 2005 edition of NBCC adopts Dynamic analysis as the standard analysis procedure for earthquake design. The equivalent static load procedure is allowed for structures in regions of low seismicity and for regular structures with periods less than 2s and less than 60m high, and for irregular structures with periods less than 0.5s and less than 20m high. All other structures require analysis using a dynamic analysis procedure. Linear dynamic analysis using the modal response spectrum method or linear time history analysis and Nonlinear analysis are specified for design.

2.3 AASHTO LRFD Bridge Specifications and ATC/MCEER Guidelines

The United States bridge design code is AASHTO LRFD Bridge Design specifications (AASHTO, 1998). AASHTO design approach was adopted in 1983. Since then, significant advances have been made in seismic design of highway bridges. In order to update the specifications to reflect recent advances in earthquake engineering,

in 1998 AASHTO proposed additional provisions to be used in conjunction with the LRFD Bridge Design Specifications. Due to the size and complexity of the new provisions, AASHTO decided that the new provisions should be published as a separate document to facilitate designers becoming familiar with the new specifications and to identify any problems before they are adopted into the LRFD guidelines. A joint team of ATC/MCEER (Applied Technology Council/ Multidisciplinary Council for Earthquake Engineering Research) accepted the task of adapting AASHTO document into a stand-alone document. In 2003, ATC/MCEER published the seismic design guide to be used as a supplement to AASHTO bridge design specifications. Changes proposed in the 2003 guidelines for the seismic design of highway bridges, are similar to the changes, in the new 2005 NBCC. The changes in ATC/MCEER 2003 guidelines include the adoption of new USGS maps, more clear performance objectives, design incentives, new soil factor and new spectral shapes. An overview of the 2000 CHBDC, AASHTO and ATC/MCEER provisions are presented in Table 2.7.

2.3.1 Definition of Seismic Hazard

Seismic Hazard in AASHTO LRFD bridge design specifications is given by contour maps, which give the acceleration coefficient, A to be used in design. The acceleration coefficient represents the peak ground acceleration (PGA) on rock at that location. The value of A is given as a ratio of gravity and it corresponds to a 10% in 50 year probability of exceedance (475 year return period).

In 1996 the USGS published new seismic hazard maps to be used in the United States (Frankel et al., 1996). The new seismic hazard values are presented as contour maps or tabulated values. The PGA values and the spectral acceleration at 0.2, 0.3 and 1

seconds are given. The USGS has presented these values at three probability levels 10% in 50 years, 5% in 50 years and 2% in 50 years. The reference ground condition is NEHRP Class B rock. The new USGS seismic hazard maps are considered in the 2003 MCEER/ATC recommendations.

2.3.2 Design Earthquake

The 2003 MCEER/ATC recommendations contain performance objectives for two design earthquakes. The maximum considered earthquake (MCE) corresponds to a 2% in 50 year probability of exceedance level. The MCE event is considered to be a rare event and the structure is expected to survive the event but may sustain significant damage. The expected earthquake (EE) has a 50% in 75 year probability of exceedance level, and the structure is expected to respond elastically to this ground motion.

2.3.3 Spectral Shapes

In AASHTO 1998 Bridge design specifications, an Elastic Seismic Response Coefficient is used to determine the spectral shape. The spectral shape is a function of the Acceleration Coefficient (A), defined by the USGS, the site coefficient and the period of the bridge. The spectral shape recommended in the 2003 guidelines is based on the 0.2 second and 1 second spectral acceleration for the MCE and EE events respectively. The values of the 0.2 second and 1 second spectral acceleration are determined based on the uniform hazard spectra procedure and as such, both values have the same probability of exceedance. However the response spectra created using these values does not have a uniform probability of exceedance at each period value. The spectral shape used in the 2003 guidelines increases in the short period range to a plateau level and then decreases in the long period range. The spectral shape used is the same in all areas of the United

States. A more detailed review of the method of defining the seismic hazard and spectral shape is presented in Chapter 3.

2.3.4 Soil Factor

AASHTO uses a similar site classification system as that in 1995 NBCC. A comparison of the soil descriptions and factor is presented in Table 2.2. The 2003 ATC/MCEER guidelines adopt the site class and site factor recommended by NEHRP in 1994. The definition of the site categories used in this method is presented in Table 2.3. The reference ground condition considered in the 2003 ATC/MCEER guidelines is site Class B, rock. Whereas in the 2005 NBCC the reference ground condition is site class C, soft rock. Therefore the F_a and F_v values used in design by the procedures of the two codes are slightly different. The values of F_a and F_v used in the 2003 ATC/MCEER guidelines are presented in Table 2.8 and Table 2.9.

2.3.5 Importance Classification and Performance Objectives

AASHTO LFRD Bridge design specification has specified the designation of an Importance Classification (IC). Essential bridges are classified as IC=I, all other bridges are classified as IC=II. Using the Importance Classification and the Acceleration Coefficient (A), the bridge is assigned to a Seismic Performance Category (SPC), ranging from A to D. The seismic performance categories are presented in Table 2.10. The seismic performance category of a bridge dictates which analysis procedure needs to be followed and the seismic design requirements. The 2003 guidelines for the Seismic Design of Highway Bridges uses Seismic Performance objectives based on Life Safety or Operational criteria. The performance objectives are presented in Table 2.11. The bridge is assigned a Seismic Hazard Level from I to IV depending on the value of F_v , S_1 and F_a .

S_s , where S_1 and S_s correspond to the 1.0 second and 0.2 second spectral acceleration. The seismic hazard levels are presented in Table 2.12. The seismic analysis and design procedure (SDAP) and the seismic design requirements (SDR) are found using the seismic hazard level and the seismic performance objective of the bridge. The SDAP and SDR of the 2003 ATC/MCEER guidelines are presented in Table 2.13.

2.3.6 Analysis and Design Methods

AASHTO recommends four analysis procedures 1) Uniform Load Method 2) Single-Mode Spectral Method 3) Multimode Spectral Method 4) Time History Method. Single Span Bridges or Bridges classified as SPC A do not require seismic analysis.

1) The Uniform Load method is an equivalent static procedure, which determines the uniformly distributed static seismic loading, P_e defined as follows:

$$P_e = \frac{C_s W}{L} \quad \text{where,}$$

$$T = 2\pi \sqrt{\frac{W}{gK}}$$

$$K = \frac{p_o L}{V_{(s, \max)}}$$

$$W = \int w(x) dx .$$

$V_{(s, \max)}$ is the maximum static displacement of the bridge due to an arbitrary uniform lateral load, p_o . The procedure is applied in both the transverse and longitudinal directions.

2) The Single-mode spectral method is based on the fundamental mode of vibration in the transverse or longitudinal direction. The mode shape is found by applying a uniform load

to the structure and calculating the corresponding deformed shape. The natural period is found by conservation of energy, by equating the maximum potential and kinetic energy.

The equivalent static seismic loading $p_e(s)$ is defined as follows:

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} W(x) V_s(x) \text{ where,}$$

$$\gamma = \int W(x) V_s^2(x)$$

$$\beta = \int W(x) V_s(x) dx$$

$$T = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}}$$

$$\alpha = \int V_s(x) dx$$

Where $V_s(x)$ is the deformation corresponding to p_o , p_o is an arbitrary uniform lateral load and $W(x)$ is the effective weight of the bridge.

3) The multimode spectral method is required for analysis where coupling occurs in more than one of three coordinate directions within each mode of vibration. A three dimensional model is used to represent the structure. The forces are combined using an accepted modal combination method such as the Complete Quadratic Combination method.

4) A Time history analysis is needed when a more detailed dynamic analysis of the structure is required. The time histories used should be selected with the help of the regulatory authority. When site specific time histories are not available, five spectrum-compatible time histories should be used. For inelastic time-history analysis the R-factor should be taken as 1.0 for all substructures and connections. The sensitivity of the solution to the size of the time step needs to be determined.

The analysis methods recommended for use in the 2003 ATC/MCEER seismic guidelines depend on the Seismic Hazard Level and the Seismic Demand and Performance objective (SDAP) of the structure as shown in Table 2.13. It includes the categories as follows: 1) No Dynamic analysis 2) No Seismic Demand Analysis 3) The Capacity Spectrum Method 4) Elastic Response Method 5) Elastic Response method with Displacement capacity verification and 6) Nonlinear dynamic Analysis. Single-span bridges do not require seismic analysis.

- 1) SDAP A1 and A2 do not require dynamic analysis.
- 2) SDAP B requires the use of Capacity Design principles and minimum design details are required.
- 3) SDAP C requires a seismic demand and capacity analysis. Regular structures, which respond as a single degree of freedom system, are permitted to use this method. In this method the structure is designed for non-seismic load combinations and then the design is checked for the demands from EE and MCE.
- 4) SDAP D uses the Elastic Response Spectrum Method of design, including the uniform load method and the multimode dynamic method. The use of R-factor is permitted with this SDAP.
- 5) SDAP E uses the Elastic Response Spectrum method with displacement capacity verification ('push-over' analysis). Higher R-factors are permitted with this SDAP. The displacement capacities are verified using two dimensional nonlinear static analyses in the principal structural directions.
- 6) Nonlinear Dynamic Analysis can be used but the same design checks applicable to SDAP E must be satisfied.

2.3.7 Design Incentives

The 2003 guidelines encourage the use of displacement capacity verification or ‘push-over analysis’. If a push-over analysis is performed the R factor used in design can be increased up to 50%.

2.4 Canadian Highway Bridge Design Code

The Canadian Highway Bridge Design code (CHBDC) was published in 2000. It adopts a methodology for seismic design similar to that in AASHTO, as presented in Table 2.7. The Canadian Highway Bridge Design code is based on seismic hazard given by GSC and uses design methods similar to those of AASHTO. Because of the recent developments of seismic hazard quantification CHBDC 2000 needs to be updated to adopt the new specification of seismic hazard in terms of UHS by GSC (Adams et al., 2003). However in order to incorporate the new information and practices, CHBDC 2000 method of seismic analysis and design need to be modified to incorporate these changes, much as NBCC had to update their seismic methodology from 1995 to 2005.

The Recommended LRFD guidelines published in 2003 by ATC/MCEER provide a methodology to incorporate similar changes in the definition of seismic hazard into the seismic design method used in AASHTO LRFD bridge design specifications.

2.4.1 Definition of Seismic Hazard

The seismic hazard is defined using the Zonal Acceleration Ratio (A), defined by GSC as in NBCC 1995. The probability of exceedance is 10% in 50 years.

2.4.2 Design Earthquake and Performance Objectives

Performance objectives of the 2000 CHBDC are defined for three different design earthquakes, small to moderate earthquakes, the Design Earthquake (475 year return period) and a large earthquake (1000 year return period). Three different performance categories are recognized, Lifeline, Emergency-Route and other, which correspond to the importance categories given in the code. A summary of the performance requirements is presented in Table 2.14.

2.4.3 Spectral Shapes

In the 2000 CHBDC Bridge design specifications an Elastic Seismic Response Coefficient (C_{sm}) is used to determine the spectral shape. The spectral shape is a function of the Acceleration Coefficient (A), given by GSC, the site coefficient, the importance factor and the period of the bridge. The procedure used to define the seismic hazard and spectral shape is reviewed in more detail in Chapter 3.

2.4.4 Site Effects

The soil profile types and site coefficients are the same as those in AASHTO and similar to those in NBCC 1995. The soil profile types and site factor are presented in Table 2.2.

2.4.5 Importance Classification

The Importance factor (I) is used to scale the Elastic seismic response coefficient. For lifeline bridges $I=3.0$, for emergency-route bridges $I=1.5$ and for other bridges $I=1.0$. Bridges are assigned to a seismic performance zone, from 1 to 4, based on the value of Peak Ground Acceleration and the importance category of the bridge. The seismic performance zones are presented in Table 2.15. The minimum analysis requirements are

determined based on the seismic performance zone and the importance category of the bridge. The minimum analysis requirements are presented in Table 2.16.

2.4.6 Analysis and Design Methods

CHBDC recommends four analysis procedures, the same as those recommended by AASHTO: 1) Uniform Load Method 2) Single-Mode Spectral Method 3) Multimode Spectral Method 4) Time History Method. No analysis is required for single-span bridges.

Table 2.1 Overview of Seismic Provisions used in the 1995 and 2005 NBCC		
	1995 NBCC	2005 NBCC
Probability of exceedance	10% in 50 years	2% in 50 years
Seismic Hazard parameter	PGA	UHS at 0.2, 0.5, 1.0 and 2.0 seconds
Spectral Shape	Standard spectral shape scaled to the PGA	<ul style="list-style-type: none"> ▪ $T \leq 0.2s$ $S(t)=F_a S_a(0.2)$ ▪ $T=0.5s$ $S(t)=F_a S_a(0.2)$ or $S(t)=F_v S_a(0.5)$ whichever is smaller ▪ $T=1.0s$ $S(t)=F_v S_a(1.0)$ ▪ $T=2.0s$ $S(t)=F_v S_a(2.0)$ ▪ $T \geq 4.0s$ $S(t)=F_v S_a(2.0)/2$
	The procedure used to define the spectral shape is studied in detail in Chapter 3.	
Site Classification	Site Categories 1, rock to 4, very soft soil, based on qualitative descriptions. * Table 2.2	Site Class A, hard rock to F, vulnerable soil based on shear wave velocity, standard penetration resistance or undrained shear strength. * Table 2.3
Foundation Factor	F, from 1.0 to 2.0 * Table 2.2	F _a or F _v * Table 2.4 and 2.5
Importance Factor	I=1.0 I=1.3 for schools I=1.5 post-disaster buildings	For ULS I=0.8 low I=1.0 normal I=1.3 high I=1.5 post-disaster
Minimum lateral seismic force	$V = \left(\frac{V_e}{R} \right) U$ $V_e = v_s I F W$	$V = \frac{S(T_a) M_v I_e W}{R_o R_d} \geq \frac{S(2.0) M_v I_e W}{R_o R_d}$ <p>Where $R_d \geq 1.5$</p> $V = \frac{2S(0.2) I_e W}{3 R_o R_d}$
Methods of Analysis	Equivalent static force procedure or dynamic analysis	Dynamic Analysis such as modal response spectrum method, linear time history method or non-linear dynamic analysis method or in certain limited cases the equivalent static force procedure.
Higher mode effects	No explicit provision	M _v factor used for higher mode effects and J the base overturning moment reduction factor.

Table 2.1 (Continued) Overview of Seismic Provisions used in the 1995 and 2005 NBCC				
R factors (Some typical examples)	Type of Lateral Force-Resisting System	R Factor 1995- used with U=0.6	Rd Factor 2005	Ro Factor 2005
	Steel Ductile MR Frame	4.0	5.0	1.5
	Steel Ductile Braced Frame	3.0	3.0	1.3
	Steel Braced frame with nominal Ductility	2.0	2.0	1.3
	RC Ductile MR Frame	4.0	4.0	1.7
	RC Ductile Wall systems	3.5	3.5	1.6
	RC MR frame nominal Ductility	2.0	2.0	1.4
U factor	U=0.6		U factor replaced by Ro factor	

Table 2.2 Soil Classification and Site Coefficient 1995 NBCC and 2000 CHBDC/ AASHTO				
Soil Profile Type	1995 NBCC Type and Dept of Rock and Soil Measured from the Foundation or Pile Cap Level	F	2000 CHBDC , 2002 AASHTO Division I-A Seismic Design	Site Coefficient
1	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15m deep	1.0	Rock of any characteristic, $V_s > 750\text{m/s}$ or stiff soil depth $< 60\text{m}$, and the soil types overlying rock are stable deposits of sands, gravels or stiff clays	1.0
2	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15m deep	1.3	Stiff clay or deep cohesionless soil depth $> 60\text{m}$ and the soil types overlying rock are stable deposits of sands, gravels or still clays.	1.2
3	Very loose and loose coarse-grained soils with depth greater than 15m	1.5	Soft to medium-stiff clay and sands, 9m or more soft to medium-stiff clay with or without intervening layers of cohesionless soil.	1.5
4	Very soft and soft fine-grained soils with depth greater than 15m	2.0	Soft clay or silt $> 12\text{m}$, $V_s < 150\text{m/s}$, might include loose natural or non-engineered fill.	2.0

Table 2.3 2005 NBCC Soil Profile / 2003 MCEER/ATC Site Classification (Based on NEHRP recommendations)		
Site Class	Soil Profile Name	Shear Wave Velocity, V_s (m/s), SPT N60, Undrained Shear Strength S_u
A	Hard Rock	$V_s > 1500$
B	Rock	$760 < V_s \leq 1500$
C	Very Dense soil and Soft Rock	$360 < V_s < 760$, $N_{60} > 50$, $S_u > 100\text{kPa}$
D	Stiff Soil	$180 < V_s < 360$, $15 \leq N_{60} \leq 50$, $50 < S_u \leq 100\text{kPa}$
E	Soft Soil	$V_s < 180$, $N_{60} < 15$, $S_u < 50\text{kPa}$
F	Others-Site Specific Evaluation Required	Liquefiable soils, sensitive clays, unstable soils, peat, high plastic clays

Table 2.4 NBCC F_a Values					
Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9

Table 2.5 NBCC F_v Values					
Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7

Table 2.6 2005 NBCC Higher Mode Factor and Base Overturning factor					
$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting Systems	M_v For $T_a \leq 1.0$	M_v For $T_a \geq 2.0$	J For $T_a \leq 0.5$	J For $T_a \geq 2.0$
< 8.0	Moment resisting frames or “coupled walls” ⁽³⁾	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems ⁽⁴⁾	1.0	1.2	1.0	0.7
≥ 8.0	Moment resisting frames or “coupled walls” ⁽³⁾	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems ⁽⁴⁾	1.0	2.5	1.0	0.4

Table 2.7 Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications			
	2000 Canadian Highway Bridge Design Code	2002 AASHTO Standard Specifications	2003 ATC/MCEER Recommended LRFD Guidelines
Probability of Exceedance	10% in 50 years.	10% in 50 years.	10%, 5% and 2% in 50 years.
Seismic Hazard Parameter	Peak Ground Acceleration	Peak Ground Acceleration	0.2 second and 1.0 second spectral acceleration.
Importance Category	<ul style="list-style-type: none"> ▪ Lifeline bridges (designed for 1000 year return period) I=3.0 ▪ Emergency-route bridges I=1.5 ▪ Other bridges I=1.0 	<ul style="list-style-type: none"> ▪ Essential Bridges ▪ Other Bridges 	Performance Objective <ul style="list-style-type: none"> • Life Safety • Operational Two Design ground motions <ul style="list-style-type: none"> • Maximum Considered Earthquake • Expected Earthquake
Seismic Performance Zone	Zones 1 to 4 based on PGA and importance category	Seismic Performance Category A to D based on PGA and the importance Classification.	Seismic Hazard level 1 to 4 based on the values of FvSl or FaSs
Analysis Requirements	Depending on seismic zone, number of spans, regularity and importance category No analysis required, uniform load method, single mode spectral method, multimode spectral method, time-history analysis.	Depending on seismic performance category, regularity and, number of spans no analysis required, uniform load method, single-mode spectral method, multimode spectral method and time-history method.	Seismic design and Analysis procedure and Seismic Design Requirement based on the Seismic Hazard level and the Performance objective.
Soil Factors	Range from 1.0 to	Range from 1.0 to 2.0	Range from 0.8 to 2.5

	2.0 based on soil type.	based on soil type.	and depend on the soil type and the short and long period spectral acceleration.										
Table 2.7 (Continued) Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications													
Elastic Seismic Response Coefficient	$C_{sm} = \frac{1.2AIS}{T_m^{2/3}} \leq 2.5AI$, for the mth mode of vibration -Where $T > 4.0s$ $C_{sm} = \frac{3AIS}{T_m^{4/3}}$ -Soil profile type 3 and 4 where $A \geq 0.3$, $C_{sm} \leq 2AI$ -Soil profile type 3 and 4 where $T < 0.3s$ $C_{sm} \leq AI(0.8 + 4T_m)$		$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A$, for the mth mode of vibration -Where $T > 4.0s$ $C_{sm} = \frac{3AIS}{T_m^{4/3}}$ -Soil profile type 3 and 4 where $A \geq 0.3$, $C_{sm} \leq 2A$ -Soil profile type 3 and 4 where $T < 0.3s$ $C_{sm} \leq A(0.8 + 4T_m)$		Sds=FaSs, where Ss is the 0.2s spectral acceleration Sdl=FvSl, where Sl is the 1.0 s spectral acceleration Ts=Sdl/Sds To=0.2Ts								
					<table><tr><td>Period</td><td>Cd</td></tr><tr><td>0-To</td><td>Sa=0.6(Sds/To)T+0.4Sds</td></tr><tr><td>To-Ts</td><td>Sa=Sds</td></tr><tr><td>>Ts</td><td>Sa=Sdl/T</td></tr></table>	Period	Cd	0-To	Sa=0.6(Sds/To)T+0.4Sds	To-Ts	Sa=Sds	>Ts	Sa=Sdl/T
	Period	Cd											
	0-To	Sa=0.6(Sds/To)T+0.4Sds											
To-Ts	Sa=Sds												
>Ts	Sa=Sdl/T												
Ductility factors	Substructure (Some typical examples)	R Factor	R Factor 2002	R Factor SDAP D *max.	R Factor SDAP E *max.								
	RC Piles-Vertical	3.0	3.0	4.0*	6.0*								
	Single Columns	3.0	3.0	4.0*	6.0*								
	Steel or composite piles-Vertical	5.0	5.0	4.0*	6.0*								
	Multiple Column Bent	5.0	5.0	4.0*	6.0*								
	Connection Column to superstructure	1.0	1.0	0.8	0.8								
	Expansion joint within span	0.8	0.8	0.8	0.8								
Design Approach	Depending on Seismic Performance Category: Uniform		Depending on Seismic Performance Category: Uniform	Depending on the Seismic Hazard Level and the Performance									

	Load Method, Single Mode Spectral Method, Multimode Spectral method or Time History analysis shall be performed.	Load Method, Single Mode Spectral Method, Multimode Spectral method or Time History analysis shall be performed.	objective of the structure: No Dynamic analysis, No Seismic Demand Analysis, Capacity Spectrum Method, Elastic Response Method or Displacement capacity verification shall be performed.
Table 2.7 (Continued) Comparison of 2000 CHBDC, AASTO and 2003 ATC/MCEER specifications			
Design Incentives	None.	None.	R-Factors increase up to 50% when a pushover/displacement capacity verification is performed.

Table 2.8 2003 ATC Fa Values					
Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Table 2.9 2003 ATC Fv Values					
Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	(1)	(1)	(1)	(1)	(1)

Table 2.10 AASHTO Seismic Performance Category		
Acceleration Coefficient	Essential Bridges	Other Bridges
$A \leq 0.09$	A	A
$0.09 < A \leq 0.19$	B	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

Table 2.11 ATC/MCEER 2003 Guidelines for the Seismic Design of Highway Bridges Design Earthquakes and Seismic Performance Objectives			
Probability of Exceedance for Design Earthquake Ground Motions	Service/Damage Level	Performance Objective	
		Life Safety	Operational
Maximum Considered Earthquake (MCE): 3% probability of exceedance in 75 years, or 1.5X median deterministic value	Service	Significant Disruption	Immediate
	Damage	Significant	Minimal
Expected Earthquake (EE): 50% probability of exceedance in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to none

Table 2.12 ATC/MCEER 2003 Guidelines for the Seismic Design of Highway Bridges Seismic Hazard Levels-Based on MCE		
Seismic Hazard Level	Value of $F_v S_1$	Value of $F_a S_s$
I	$F_v S_1 \leq 0.15$	$F_a S_s \leq 0.15$
II	$0.15 < F_v S_1 \leq 0.25$	$0.15 < F_a S_s \leq 0.35$
III	$0.25 < F_v S_1 \leq 0.40$	$0.35 < F_a S_s \leq 0.60$
IV	$0.40 < F_v S_1$	$0.60 < F_a S_s$

For site class E and F: F_v and F_a are less than 2.4 and 1.6 when $S_1 < 0.1$ and $S_s < 0.25$

Table 2.13 2003 ATC/MCEER Seismic Design Requirements (SDR)				
Seismic Hazard Level	Life Safety		Operational	
	SDAP	SDR	SDAP	SDR
I	A1	1	A2	2
II	A2	2	C/D/E	3
III	B/C/D/E	3	C/D/E	5
IV	C/D/E	4	C/D/E	6

Table 2.14 2000 CHBDC Performance Requirements			
Return Period	Bridge		
	Lifeline	Emergency-Route	Other
Small to Moderate Earthquake	All traffic Immediate use	All traffic Immediate use	All traffic Immediate use
Design Earthquake (475 year return period)	All traffic Immediate use	Emergency vehicles Immediate use	Repairable damage
Large Earthquake (1000 year return period)	Emergency vehicles Immediate use	Repairable damage	No collapse

Table 2.15 2000 CHBDC Seismic Performance Zones		
PHA	Emergency and Other Bridges	Lifeline Bridges
0.00-0.08	1	2
0.08-0.16	2	3
0.16-0.23	3	3
0.23-	4	4

Table 2.16 2000 CHBDC Minimum Analysis Requirements for Multispan Bridges						
Seismic Performance Zone	Lifeline Bridges		Emergency-route Bridges		Other Bridges	
	Regular	Irregular	Regular	Irregular	Regular	Irregular
1	NA	NA	None	None	None	None
2	Multimode	Multimode	Uniform Load	Multimode	Uniform Load	Single mode
3	Multimode	Time History	Multimode	Multimode	Uniform Load	Multimode
4	Multimode	Time History	Multimode	Multimode	Single Mode	Multimode

Chapter 3

3. Seismic Hazard Parameter and Spectral Shape

3.1 1995 National Building Code of Canada

The seismic hazard at a given location is specified in NBCC 1995 by the acceleration-related seismic zone (Z_a), the velocity related seismic zone (Z_v) and the zonal velocity ratio (v). Based on the period of the structure and the ratio of Z_a / Z_v , a value for the seismic response factor (S) is determined. The values of the zonal velocity ratio (v), the acceleration related seismic zone and the velocity related seismic zone are presented in Table 3.1 for Montreal, Ottawa, Toronto and Vancouver. The plot of the seismic response factor gives the spectral shape, which is then scaled by the zonal velocity ratio to obtain the seismic hazard at a particular location. The values of S are presented in Table 3.2 and are shown graphically in Figure 3.1. The zonal velocity ratio in 1995 NBCC corresponds to a 10% in 50 year probability of exceedance. However that probability only applies to the peak acceleration value and is not uniform for all the period values. The spectral acceleration value is determined by multiplying the seismic response factor (S) by the zonal velocity ratio (v). The spectral shapes for Montreal, Ottawa, Toronto and Vancouver are presented in Figure 3.2. The spectral shapes for Montreal and Ottawa are the same.

3.2 2005 National Building Code of Canada

The 2005 NBCC adopts the uniform hazard spectra approach in defining seismic hazard. The probability of exceedance level of the UHS is taken at 2% in 50 years. The design spectral acceleration values are determined as follows:

- $T \leq 0.2s \quad S(t) = F_a S_a(0.2)$

- $T=0.5s$ $S(t)= F_a S_a (0.2)$ or $S(t)=F_v S_a(0.5)$ whichever is smaller
- $T=1.0s$ $S(t)= F_v S_a (1.0)$
- $T=2.0s$ $S(t)= F_v S_a (2.0)$
- $T \geq 4.0s$ $S(t)= F_v S_a (2.0)/2$

The fourth generation seismic hazard maps of Canada by GSC (Adams et al., 2003) provide the spectral acceleration values at 0.2, 0.5, 1.0 and 2.0 seconds and the peak ground acceleration for 650 locations throughout Canada at a 2% in 50 year probability of exceedance level. GSC provides the spectral acceleration at both 2% in 50 years and 10% in 50 years for selected Canadian cities. These values are provided for both the source models used by GSC (H or R) and at two confidence levels (50% and 84%). The Uniform Hazard Spectra for four Canadian Cities have been constructed. The cities chosen are Montreal, Ottawa, Toronto and Vancouver. These cities represent a range of seismic environments in eastern and western Canada. Montreal and Ottawa have a similar seismic environment, with moderate levels of seismicity. Toronto has a low level of seismicity and Vancouver has a high level of seismicity. The model, which produced the larger spectral acceleration at a given period, either H or R, called the Robust method, are used by GSC to define the spectra. The spectra are constructed using values corresponding to the 50% confidence level (median values). The spectral values used to generate the UHS at 2% in 50 years and 10% in 50 years are presented in Table 3.3. The spectral values in the table are obtained from an open file report by GSC (Adams et al., 2003). The values for Montreal and Ottawa are very similar. The result is that in many cases the differences between the two cities is minimal.

3.3 Comparison of the Spectral Shape of 1995 NBCC and 2005 NBCC

The spectral values specified in NBCC 1995 are plotted with the 2% in 50 year and 10% in 50 year UHS values for Montreal/Ottawa, Toronto and Vancouver in Figures 3.3, 3.4 and 3.5. For all three cities in the short period range the use of the 2% in 50 year UHS values represent an increase in the design spectral acceleration. In the case of Montreal/Ottawa and Toronto at just before the one second period the 2% in 50 year UHS values begin to drop off more quickly than NBCC 1995 spectral values. For Vancouver this trend can be observed after the one second period. Montreal/Ottawa and Vancouver, which have moderate to high seismic hazard levels have larger increases in spectral acceleration in the short period region of the 2% in 50 year UHS compared to NBCC 1995 spectral values. Toronto, which has low seismicity has a relatively smaller increase. The 10% in 50 year UHS is lower than NBCC 1995 spectral values for all three cities. However of the three cities, NBCC 1995 spectral values are closest to the 10% in 50 year UHS in Vancouver.

3.4 Canadian Highway Bridge Design Code and AASHTO

Both CHBDC 2000 and AASHTO use an Elastic Seismic Response Coefficient to define the spectral shape. The Elastic Seismic Response Coefficient, C_{sm} is defined as follows:

- $C_{sm} = \frac{1.2AIS}{T_m^{2/3}} \leq 2.5AI$, for the m^{th} mode of vibration.
- When $T > 4.0\text{s}$, $C_{sm} = \frac{3AIS}{T_m^{4/3}}$.
- For soil profile type 3 and 4 where $A \geq 0.3$, $C_{sm} \leq 2AI$.

- For soil profile type 3 and 4, for modes other than the fundamental mode where $T < 0.3$, $C_{sm} \leq AI(0.8 + 4T_m)$.
- The method used by CHBDC and AASHTO is identical with the exception that in AASHTO the I factor is not used.

The zonal acceleration ratio, A , is defined at 10% in 50 year probability of exceedance. In AASHTO specifications the importance factor is combined with the ductility factor. For the purpose of this study, the importance factor for use in CHBDC 2000 provisions is taken as $I=1.0$. However for many common structures, such as those considered later in this study, the importance factor would likely be $I=1.5$, or possibly $I=3.0$, which would increase the magnitude of the design spectrum accordingly. For the purpose of this study the site is assumed to be soil profile 1, with $S=1.0$. Figure 3.6 shows an example of the spectral curve for the various soil types in Ottawa. Figure 3.7 shows the spectral shapes of Montreal, Ottawa, Toronto and Vancouver using CHBDC 2000/AASHTO method, with the importance factor equal to 1.0.

3.5 2003 ATC/MCEER

The 2003 ATC/MCEER guidelines for the seismic design of highway bridges considers two categories of design earthquake, the expected earthquake (EE), which corresponds to a 50% in 75 year probability of exceedance and the maximum considered earthquake (MCE), which corresponds to a 3% in 75 year probability of exceedance (equivalent to 2% in 50 years). For the purpose of this study, the MCE is considered for comparison with other design code procedures. To construct the design spectra, the spectral acceleration at 0.2 and 1.0 seconds are required (S_s and S_1). The USGS provides

these values in national ground motion maps with contour values or in tabular form. This method uses the site factors to define the spectral shape. S_{ds} and S_{dl} are defined for the location based on the soil type and the spectral acceleration at the locations where:

- $S_{ds} = F_a S_s$ and $S_{dl} = F_v S_l$.
- The value of T_s and T_o are defined as $T_s = S_{dl} / S_{ds}$ and $T_o = 0.2 T_s$.

The design spectra is defined as follows:

- $T=0, S_a=0.40 S_{ds}$
- $0 < T \leq T_o, S_a = 0.60 \frac{S_{ds}}{T_o} T + 0.40 S_{ds}$
- $T_o < T \leq T_s, S_a = S_{ds}$
- $T > T_s, S_a = \frac{S_{dl}}{T}$

The shape of the spectral curve for the various soil types in Ottawa is shown in Figure 3.8. The reference ground condition used is Class B rock. The reference ground condition for use with the UHS values defined by GSC is site class C. To compare the ATC method with the method previously used by CHBDC 2000 and AASHTO, the reference ground condition is taken as site class C, and the F_a and F_v values defined in 2005 NBCC are used instead of those adopted for use in the 2003 ATC/MCEER guidelines. The shape of the spectral curve for Montreal, Ottawa, Toronto and Vancouver developed using the modified 2003 ATC/MCEER method, with site class C as the reference ground condition is shown in Figure 3.9.

3.6 Comparison of Spectral Shape of CHBDC, ATC and UHS

The spectral shape of the 2000 CHBDC/AASHTO method and 2003 ATC/MCEER method are compared to the UHS at 2% in 50 years and 10% in 50 years probability of

exceedance in Figure 3.10, 3.11, 3.12 and 3.13. The 2% in 50 year UHS for Montreal, Ottawa and Toronto is higher than CHBDC 2000 curve in the short period range but at a period of 0.2 to 0.4 seconds the UHS curve drops off more quickly and CHBDC 2000 values are higher. In Vancouver the 2% in 50 year UHS is larger than CHBDC 2000 curve until a period of about 2 seconds. The 2003 ATC/MCEER spectral curve developed for the MCE with a 2% in 50 year probability of exceedance is similar in shape and magnitude to the 2% in 50 year UHS. In the short period range they are almost identical for the four cities. In the intermediate period range the 2% in 50 year UHS is larger than the 2003 ATC/MCEER values, except for Vancouver where the UHS is slightly lower. Beyond the period of about 2 seconds the 2% in 50 year UHS values are lower than the 2003 ATC values for all cities. The 10% in 50 year UHS is lower than CHBDC 2000 values for all the cities. Toronto and Vancouver show the largest increase in spectral acceleration comparing CHBDC 2000 values to the 2% in 50 year UHS values, while Montreal and Ottawa show more moderate increases.

3.7 Development of UHS Compatible Ground Motions

Simulated Ground Motion records matching the 10% and 2% in 50 year UHS are obtained from Atkinson (Atkinson, 1998). For each probability level three different suites of ground motion are provided, one for eastern Canada, one for western Canada and one for the Cascadia earthquake scenario. The 10% in 50 year ground motions provided two trials for each magnitude distance combination and the 2% in 50 year ground motions provided four trials for each magnitude distance combination. In this study, the ground motions for eastern Canada are used to match the UHS in Montreal,

Ottawa and Toronto, while the western Canada ground motions are used to match the UHS in Vancouver. Two time history records are used to match each UHS. One time history record is required for the short period range (0.1-0.5s) and another for the long period range (0.5-5s). Usually the short period range is best matched by an earthquake of moderate magnitude size and closer distance range, such as M5.5 at 20km, whereas the long period range is best matched by another earthquake of larger magnitude at a greater distance, such as M7 at 50km. The time history records are scaled up or down to best fit the target UHS. Recommended scaling factors range from 0.6 to 1.3, however for this study a range of 0.5-2 is used, in increments of 0.1 (Atkinson et al., 1998). The minimum variance criterion is used to select the time history and scale factor which best fits the target UHS (Phung, 2005).

Table 3.4 and 3.5 show the magnitude, distance, scale factor, maximum acceleration (unscaled) and duration of the ground motions matching the UHS for each city for 10% and 2% in 50 year probability of exceedance in the short and long period ranges. The ground motions matching the 10% in 50 year and 2% in 50 year UHS in Montreal and Ottawa are the same. The time history ground motions compatible with the 10% in 50 year and 2% in 50 year UHS in the short and long period range are plotted in Figures 3.14 to 3.17. The accerlogram and peak spectral acceleration for each ground motion recording are shown in Figures 3.18 to 3.30. It is noted that the duration of a short period ground motion record is shorter than the duration of a corresponding long period ground motion record. The ground motion records for Montreal/ Ottawa and Toronto have similar characteristics. In the short period range for 10% and 2% in 50 year probability of exceedance, the same ground motion recording is used by applying different scaling

factors for Montreal/Ottawa and Toronto. In the long period range, Montreal and Ottawa use the same ground motions and scaling factors. The ground motions for use in Vancouver show larger energy input than the ground motions for eastern Canada.

3.8 Summary

- Table 3.6 shows the relative increase of the UHS compared to the previous NBCC and CHBDC code values.
- NBCC show fairly uniform changes for Montreal, Ottawa, Toronto and Vancouver. The 10% in 50 year UHS is less than the code values for all period ranges. The 2% in 50 year UHS is greater than the code values in the short period range. In the intermediate range in Eastern cities, the 2% in 50 year UHS is closer to the code values, while for Vancouver the UHS is still greater than the code values in this period range. In the long period range the 2% in 50 year UHS is smaller than NBCC code values.
- CHBDC 2000 shows more variability between the Eastern and Western cities. The 10% in 50 year UHS is smaller than the code values for Montreal, Ottawa and Toronto. However in the short period range, for Vancouver the 10% in 50 year UHS is almost equal to CHBDC 2000 spectral values. The 2% in 50 year UHS is greater than CHBDC 2000 code values in the short period range for all the cities. However in the intermediate period range in Montreal, Ottawa and Toronto CHBDC 2000 spectral values are approximately equal to the 2% in 50 year UHS values and in the long period range CHBDC 2000 code values are larger than the 2% in 50 year UHS values. In Vancouver the 2% in 50 year UHS is larger than the code values at all period ranges.

- The modified 2003 ATC/MCEER spectral shapes generated using GSC spectral values and using F_a and F_v values equal to 1.0 are very similar in shape and magnitude to the 2% in 50 year UHS.
- The UHS compatible ground motions for Montreal, Ottawa and Toronto are similar.
- The UHS compatible ground motions for Vancouver show a larger energy content than those in the east.

Table 3.1 1995 NBCC Seismic Design Parameters				
	v	Z_a	Z_v	Z_a/Z_v
Montreal	0.1	4	2	2
Ottawa	0.1	4	2	2
Toronto	0.05	1	0NA	
Vancouver	0.2	4	4	1

Table 3.2 1995 NBCC Seismic Response Factor		
T	Z_a/Z_v	S
≤ 0.25	>1.0	4.2
	1.0	3.0
	<1.0	2.1
$0.25 < T < 0.50$	>1.0	$4.2-8.4(T-0.25)$
	1.0	$3.0-3.6(T-0.25)$
	<1.0	2.1
≥ 0.50	All values	$1.5/T^{1/2}$

Table 3.3 UHS Values (Robust method and 50% confidence level)								
	Montreal		Ottawa		Toronto		Vancouver	
Period (s)	10% in 50yrs	2% in 50 yrs	10% in 50yrs	2% in 50 yrs	10% in 50yrs	2% in 50 yrs	10% in 50yrs	2% in 50 yrs
0-pga	20	43	20	42	8	20	26	46
0.1	29	65	28	64	11	28	44	83
0.2	29	69	28	66	11	28	51	94
0.5	13	34	12	32	5.2	14	35	64
1	5.2	14	5	13	2.2	5.5	18	33
2	1.6	4.8	1.5	4.4	0.6	1.6	8.9	17
4	0.8	2.4	0.75	2.25	0.3	0.8	4.45	9
5	0.8	2.4	0.75	2.25	0.3	0.8	4.45	9

- Spectral Acceleration Values are in % g

Table 3.4 Ground Motion records matching 10% in 50 year UHS								
10% - 50years	Montreal		Ottawa		Toronto		Vancouver	
Period Range	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s
Magnitude	5.5	7.0	5.5	7.0	5.5	7.0	6.0	7.2
Distance (km)	70	300	70	300	70	300	10	50
Scale Factor	2.0	0.8	2.0	0.8	0.8	0.5	0.5	1.0
Agmax (cm/s ²)	47.34	42.82	47.34	42.82	47.34	53.52	477.34	146.66
Duration (s)	15.5	32.07	15.5	32.07	15.5	32.07	6	17.98

Table 3.5 Ground Motion records matching 2% in 50 year UHS								
2% - 50years	Montreal		Ottawa		Toronto		Vancouver	
Period Range	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s	T $\leq 0.5s$	T > 0.5s
Magnitude	6	7.0	6	7.0	6	7.0	6.5	7.2
Distance (km)	50	50	50	50	50	100	30	40
Scale Factor	1.5	0.6	1.5	0.6	0.6	0.5	1.0	0.5
Agmax (cm/s ²)	198.15	619.76	198.15	619.76	198.15	237.81	526.83	496.02
Duration (s)	12.4	20.55	12.4	20.55	12.4	23.07	8.53	16.64

Table 3.6 Relative value of 2% in 50 year and 10% in 50 year UHS spectral acceleration in the short(<0.5s), intermediate (0.5-1.0s) and long period ranges (>1.0s) with respect to the previous code values

	1995 NBCC			
	10% in 50 years	2% in 50 years		
	All period ranges	Short period	Intermediate period	Long period
Montreal	Relative decrease	Increase	Increase to Equal	Relative decrease
Ottawa	Relative decrease	Increase	Increase to Equal	Relative decrease
Toronto	Relative decrease	Increase to Equal	Increase to Equal	Relative decrease
Vancouver	Relative decrease	Increase	Increase	Relative decrease
	2000 CHBDC			
	10% in 50 years	2% in 50 years		
	All Period Ranges	Short period	Intermediate period	Long period
Montreal	Relative decrease	Increase to Equal	Equal-Decrease	Relative decrease
Ottawa	Relative decrease	Increase to Equal	Equal-Decrease	Relative decrease
Toronto	Relative decrease	Increase	Equal	Relative decrease
Vancouver	Equal-Decrease	Increase	Increase	Increase

Figure 3.1 1995 NBCC Seismic Response Factor

Figure 3.2 Spectral Shapes 1995 NBCC

Figure 3.3 Montreal/Ottawa Spectral Shapes

Figure 3.4 Toronto Spectral Shapes

Figure 3.5 Vancouver Spectral Shapes

Figure 3.6 Ottawa 2000 CHBDC, 2002 AASHTO Method

Figure 3.7 AASHTO/CHBDC Spectra Soil Type 1

Figure 3.8 Ottawa 2003 ATC Method

Figure 3.9 2003 ATC Method, Site Class C

Figure 3.10 Ottawa Spectral Shapes

Figure 3.11 Montreal Spectral Shapes

Figure 3.12 Toronto Spectral Shapes

Figure 3.13 Vancouver Spectral Shapes

Figure 3.14 Montreal/Ottawa UHS Matched Time History Records

Figure 3.15 Toronto UHS Matched Time History Records

Figure 3.16 Vancouver UHS Matched Time History Records

Figure 3.17 Montreal/Ottawa 10% In 50 Year Short Period Time History Record

Figure 3.18 Montreal/Ottawa 10% In 50 Year Long Period Time History Record

Figure 3.19 Montreal/Ottawa 2% In 50 Year Short Period Time History Record

Figure 3.20 Montreal/Ottawa 2% In 50 Year Long Period Time History Record

Figure 3.21 Toronto 10% In 50 Year Short Period Time History Record

Figure 3.22 Toronto 10% In 50 Year Long Period Time History Record

Figure 3.23 Toronto 2% In 50 Year Short Period Time History Record

Figure 3.24 Toronto 2% In 50 Year Long Period Time History Record

Figure 3.25 Vancouver 10% In 50 Year Short Period Time History Record

Figure 3.26 Vancouver 10% In 50 Year Long Period Time History Record

Figure 3.27 Vancouver 2% In 50 Year Short Period Time History Record

Figure 3.28 Vancouver 2% In 50 Year Long Period Time History Record

Chapter 4

4. Displacement Ductility and Force Reduction Factors

4.1 General

Earthquakes are rare but significant events for consideration in structural design. There are many uncertainties in the estimation of the maximum seismic loading effects imparted by the most severe earthquake expected over the service life of a structure. The seismic design strategy of the 2005 NBCC is to adopt a design earthquake with the same return period as the desired probability of failure for the structure, notably 1 in 2500 years. The likelihood of such an event happening during the typical 75 year design service life of a bridge is low while the load induced by an earthquake of that magnitude is high. The generally accepted seismic design philosophy therefore is to design structures expected to behave inelastically during rare earthquake events to reduce costs. For certain regions of high seismicity, structures may be designed to behave elastically during more frequent moderate magnitude earthquakes. When the structure responds inelastically, it is expected that a certain amount of damage is sustained. The performance objectives of most design guides require that the damage suffered by the structure would not lead to collapse for the case of a major severe earthquake of low probability of occurrence, and that the location of damage should be such that the structure can be repaired for the case of a moderate earthquakes. The earthquake resisting elements of a structure designed to behave this way must have adequate ductility to sustain repeated inelastic deformation reversal cycles without failure.

The main earthquake resisting elements of bridges are the substructure elements. In seismic design of bridges, force reduction factors are used to design the substructure

elements, such that they behave as the primary energy dissipating agents in the bridge.

Design forces in the substructure elements, are found by performing an elastic analysis of the structure and then dividing the forces by an appropriate response modification factor,

R. With proper detailing design of the bridge substructure elements, such as pier columns, walls or bents, the bridge structure would have sufficient reserve ductility capacity to withstand the strong shaking of major earthquakes without collapse.

4.2 Displacement Ductility and Force Reduction Factors

Ductility is a measure of the amount of inelastic deformation a member or section can sustain without collapse or failure during dynamic responses. It is quantified relative to the member or section's yield deformation. Ductile behaviour of a structural element implies that it can sustain large inelastic action beyond yield without collapse. Ductility can be measured by means of the deformation quantities of displacement, rotation or curvature. The displacement ductility demand is defined as

$$\mu_{\Delta} = \frac{\Delta_{\max}}{\Delta_y}$$

where $\Delta_{\max}$ is the maximum displacement and Δ_y is the yield displacement. For bridges the displacement ductility is evaluated with respect to critical substructure elements such as the columns or piers. The rotational ductility demand

$$\mu_{\theta} = \frac{\theta_{\max}}{\theta_y} \text{ as well as the curvature ductility demand,}$$

$$\mu_{\phi} = \frac{\phi_{\max}}{\phi_y},$$

where $\theta_{\max}, \phi_{\max}$ denote the maximum rotation and curvature of the structural element under dynamic responses and θ_y, ϕ_y are the corresponding yield quantity, are also

important indicators, which characterize the performance of a structure during earthquakes. A ductility demand values less than 1.0, represents elastic behaviour in the bridge.

The response modification factor is related to the displacement ductility. For short period structures it has been found that because the structure experiences nearly the same acceleration as the ground that there is no corresponding reduction in force due to ductility and the value of R approaches 1.0. Structures in the intermediate period range experience pulses, which are shorter than the fundamental period of vibration of the structure and the resulting value of response reduction factor can be approximated by, $R = \sqrt{2\mu - 1}$. For longer period input motions this relation is reduced to $R = 2(1 - 0.5\mu)$ (Priestley et al., 1996). However due to the complex behaviour of a bridge, its geometry and the site effects, approximations are usually considered in the relationship between displacement ductility and the force reduction factor. Generally the force reduction factor decreases in the short period range and increases to a constant level in the intermediate and long period range where it is considered equal to the displacement ductility. At the period of $T=0$ the force reduction factor and displacement ductility factor are equal to 1.0, then it gradually increases until $R = \mu_\Delta$, as follows,

$$R = 1 + 0.67(\mu_\Delta - 1) \frac{T}{T_o} \leq \mu_\Delta,$$

where T_o is the period at peak elastic spectral response (Priestly et al., 1996). In earthquake engineering design practice, the force reduction factors used in most bridge design codes are based on test data and experience.

4.3 CHBDC/AASHTO Force Reduction Factors

The R factor in the 2000 CHBDC is used to reduce the design force obtained from an elastic analysis of ductile substructure elements. In Seismic Performance Zones 2, 3 and 4 these R factor values may only be used when the specified design and detailing requirements are met to ensure the elements have sufficient ductility. The R factors used by CHBDC 2000 are the same as those used in AASHTO provisions. In AASHTO the response modification factor may be used for bridges classified as seismic performance categories B, C or D. CHBDC 2000 and AASHTO R values are presented in Table 4.1.

4.4 2003 ATC/MCEER Response Modification Factors

The 2003 ATC/MCEER guidelines for bridges that are designed using seismic design and analysis procedures D and E permit the use of the R factor. However the value of R is limited by the equation:

$$R = 1 + (R_b - 1) \frac{T}{1.25T_s} \leq R_b ,$$

where $T_s = S_{dl}/S_{ds}$ and T is the period of vibration of the bridge. The R values are presented in Table 4.2. If SDAP E (displacement capacity verification-‘push-over’ analysis) is used, the design forces can be further reduced but should not be lower than 70% of the original design force.

4.5 Development of Displacement Ductility Spectra

The response of a single degree of freedom oscillator to time history ground motions can be found using the NONSPEC computer program. The NONSPEC program computes the inelastic response spectra of a single degree of freedom system to dynamic loads such as impulse loads, harmonic functions or seismic excitation. The equation of

motion for horizontal earthquake ground motions is expressed as follows (Mahin et al., 1983):

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = -M\ddot{u}_g(t) \quad (4.1)$$

Where M is the mass, C is the system's damping and K is the system's stiffness.

Replacing $C=2\omega\xi M$ and $K=\omega^2 M$ and then dividing equation 4.1 by M leads to

$$\ddot{u}(t) + 2\omega\xi\dot{u}(t) + \omega^2 u(t) = -\ddot{u}_g(t) \quad (4.2)$$

Replacing $u(t)=R(t)/K$ and substituting $K=\omega^2 M$ and dividing by u_y equation 4.2

becomes:

$$\frac{\ddot{u}(t)}{u_y} + 2\omega\xi\frac{\dot{u}(t)}{u_y} + \frac{R(t)}{Mu_y} = -\frac{\ddot{u}_g(t)}{u_y} \quad (4.3)$$

Taking $\frac{R(t)}{Mu_y} = \omega^2 \rho(t)$ and $\frac{\ddot{u}_g(t)}{u_y} = \omega^2 (\frac{M}{R_y})\ddot{u}_g(t)$ and defining a non-dimensional

parameter η equal to the system's yield strength relative to the maximum inertia force of an infinitely rigid system, where η is called the strength-inertia force ratio and is defined as follows:

$$\eta = \frac{R_y}{M\ddot{u}_{g\max}}$$

The equation 4.3 becomes:

$$(\ddot{\mu}(t)) + 2\omega\xi\dot{\mu}(t) + \omega^2 \mu(t) = -\frac{\omega^2}{\eta} \frac{\ddot{u}_g(t)}{\ddot{u}_{g\max}} \quad (4.4)$$

The displacement ductility response reflects the characteristics of the structural behaviour. Displacement Ductility demand greater than 1.0 indicate inelastic behaviour, while values less than 1.0 indicates elastic response. The generation of time history

ground motions compatible with the 2% in 50 year and 10% in 50 year UHS are presented in Chapter 3.

4.5.1 Constant Strength-Inertia Ratio Displacement Ductility Spectra

Displacement ductility curves for Montreal, Ottawa, Toronto and Vancouver for the 2% in 50 year and 10% in 50 year UHS compatible ground motions have been calculated using NONSPEC as shown in Figures 4.1 to 4.4. The displacement ductility curves period range from 0 to 2 seconds, which covers the range of periods of typical bridge structures are presented. The displacement ductility is plotted for constant η values of 0.2, 0.5 and 1.0. The value of η is dependant on the strength of the system, its mass and the maximum acceleration of the ground motion. The maximum ground acceleration is different for each ground motion used to match the UHS in each city. Therefore an η value of 0.2 in Montreal, Ottawa, Toronto and Vancouver, means that the strength of the system would be different for each city (except for the cases where the ground motion for two cities is identical, such as Montreal and Ottawa), a constant η value for each of the four cities implies four different structures/systems. The UHS for Montreal and Ottawa are the same and they will be discussed together.

The analysis is carried out for an elasto-perfectly plastic, single degree of freedom system. An examination of the trend in displacement ductility and the magnitude of the displacement ductility values relative to 1.0 (indicating elastic response) are useful.

Figure 4.1 shows the positive displacement ductility demand for Montreal, Ottawa, Toronto and Vancouver subjected to the UHS compatible ground motion for 10% in 50 years in the short period range. The lower strength-inertia ratio systems, $\eta=0.2$, have displacement ductility greater than 10.0, in the short period range. However

the displacement ductility drops off rapidly after 0.3 seconds and become elastic after about one second. In the intermediate strength-inertia ratio range, $\eta=0.5$ the maximum displacement ductility value is about 4.0 for Vancouver and 2.0 for Montreal/Ottawa and Toronto. The displacement ductility for Toronto and Montreal/Ottawa has become elastic by 0.6 second period while the displacement ductility value for Vancouver becomes elastic after about two seconds. The displacement ductility for the high strength-inertia ratio, $\eta=1.0$ are equal to approximately 1.0 in the short period range but become elastic at a period of 0.3 seconds.

Figure 4.2 shows the positive displacement ductility demand for the four cities in response to the UHS compatible 10% in 50 year long period ground motions. The displacement ductility is larger for both the low strength-inertia ratio systems and intermediate strength-inertia ratio systems and is greater than 1.0 until after the 1.0 second period. The higher strength-inertia ratio system is inelastic in the lower period range but becomes elastic by the one second period mark.

Figure 4.3 shows the positive displacement ductility demand for the four cities in response to the UHS compatible 2% in 50 year short period ground motions. The displacement ductility for Vancouver for the low and intermediate strength-inertia ratio systems is greater than 1.0 for all period ranges, indicating inelastic response at all periods. Montreal, Ottawa and Toronto low strength-inertia ratio systems display inelastic behaviour up to the one second period. In the intermediate strength-inertia ratio range their behaviour becomes elastic by the 0.5 second period. For the higher strength-inertia ratio systems all cities begin with displacement ductilities in the inelastic range,

however by 0.4 second period the values of Montreal, Ottawa and Toronto have become elastic and by the one second period the values of Vancouver have become elastic.

Figure 4.4 shows the positive displacement ductility demand for the four cities in response to the UHS compatible 2% in 50 year long period ground motions. The low strength-inertia ratio system responds inelastically for Vancouver, while the response for Montreal/Ottawa and Toronto drops to a displacement ductility demand equal to one after the one second period. The intermediate strength-inertia ratio systems begin in the inelastic range however the values of the response to Montreal/Ottawa and Toronto become elastic by 0.9 second period, while the response to Vancouver remains inelastic until the two second period. The high strength-inertia ratio system begin in the inelastic range, the response values of Montreal/Ottawa and Toronto have become elastic by 0.6 and 0.5 second periods respectively and the response values of Vancouver become elastic by the one second period.

4.5.2 Constant Strength Displacement Ductility Spectra

In Figures 4.5 to 4.7 the positive displacement ductility demand for each city to ground motions compatible with both probability of exceedance levels in the short and long period range are plotted. In these Figures the strength of the system has been kept constant for each ground motion. The mass, stiffness and damping for each system are identical and are based on a single column reinforced concrete bridge structure. The mass of the system is taken as $M=1.5 \times 10^6$ kg, the period $T=0.5$ seconds and the damping $C=5\%$ critical. Using these values, η values for each ground motion could be calculated. The analysis is performed for an elasto-perfectly plastic, single degree of freedom system, using the calculated η values. The method of using site specific η values

provides more valuable information about the ground motions than comparing constant η values because it is possible to compare the response of a particular system to varying input ground motions.

Figure 4.5 show the positive displacement ductility demand for Montreal and Ottawa. The UHS curves for these cities are nearly identical and therefore the ground motions used to match the UHS are the same and these cities will be discussed together. In the short period range, for the 2% in 50 year probability level the maximum displacement ductility demand is greater than 10. The value quickly drops, such that by the 0.6 second period for the short period ground motion and one second period for the long period ground motion the value has become elastic. For the 10% in 50 year probability ground motion, the long period ground motion is entirely in the elastic range. While the short period ground motion has a maximum of about 1.5 but quickly decreases such that the displacement ductility is less than 1.0 by the 0.2 second period.

Figure 4.6 shows the positive displacement ductility demand for Toronto. The maximum displacement ductility is 2.5 for the 2% in 50 year short and long period ground motions. The displacement ductility becomes elastic by 0.5 second period for the short period ground motion and 0.6 second period for the long period ground motion. Both the short and long period ground motion for the 10% in 50 year probability level respond in the elastic range.

Figure 4.7 shows the positive displacement ductility demand for Vancouver. The maximum displacement ductility is over 10.0 for both the short and long period ground motion in response to the 2% in 50 year probability level. The short period compatible ground motion shows greatest inelastic behaviour and remains inelastic throughout the

period range. The long period compatible ground motion drops below a displacement ductility demand of 4.0 by the 0.3 second period and becomes elastic by the one second period. The 10% in 50 year compatible ground motions have maximum values of about 3.0 and 4.0 for the long and short period ranges respectively, the values become elastic at the 1.5 second period for the short period ground motion and two second period for the long period ground motion.

4.6 Summary

- Referring to Figures 4.1 to 4.4, constant strength-inertia force ratio:
- Low strength-inertia ratio systems $\eta=0.2$, experienced inelastic response up to a period of one second for both 10% in 50 year and 2% in 50 year short and long period compatible ground motions. However the response to the Vancouver ground motion is inelastic over the entire period range except for the case of the 10% in 50 year long period ground motion, which became elastic at a two second period.
- Intermediate strength-inertia ratio systems $\eta=0.5$, experienced more moderate inelastic response, which often became elastic before the one second period for Montreal, Ottawa and Toronto. However the response to the Vancouver ground motion is inelastic up to the two second period.
- High strength-inertia ratio systems $\eta=1.0$, experienced low levels of inelastic response for periods less than 0.5 seconds, after which the response are in the elastic range. However Vancouver remained in the inelastic range until the one second period for the 2% in 50 year ground motions.

- The largest positive displacement ductility demand occurred in the low strength-inertia ratio systems $\eta=0.2$, in the short period range.
- There is a significant difference in the maximum positive displacement ductility demand between the $\eta=0.2$, $\eta=0.5$ and $\eta=1.0$ strength-inertia ratio systems. The low strength system has a significantly larger response, followed by the intermediate strength systems. The high strength systems had little to no inelastic response.
- Vancouver had the highest positive displacement ductility demand, particularly in response to the 2% in 50 year ground motions.
- Referring to Figures 4.5 to 4.8, constant strength values:
- The short period displacement ductility demand (<0.5 seconds) for the 2% in 50 year ground motions is significantly greater than that for the 10% in 50 year ground motions.
- The magnitude of the displacement ductility demand for Vancouver is the largest followed by Montreal/Ottawa and finally Toronto.
- Montreal/Ottawa have elastic response to the 10% in 50 year long period ground motion and elastic response to the 10% in 50 year short period ground motion after a period of 0.4 seconds. Large inelastic response to the 2% in 50 year ground motions for period values less than 0.3 seconds, followed by more moderate ductility demands and a return to elastic behaviour by the one second period.

- Toronto has elastic response to both 10% in 50 year ground motions. For the 2% in 50 year ground motion low ductility demand in the short period range is followed by a return to elastic behaviour after a period of 0.6 seconds.
- Vancouver has high ductility demand in response to the 2% in 50 year short period compatible ground motion and the response remains inelastic throughout the period range. The 2% in 50 year long period compatible ground motion has a more moderate ductility demand, which drops into the elastic range by a period of two seconds. Both 10% in 50 year ground motions have moderate ductility displacement responses, which become elastic after the one or two second periods.

4.7 Conclusions

- Montreal, Ottawa and Toronto respond elastically (or with very low inelastic behaviour) to the 10% in 50 year ground motions.
- Montreal, Ottawa and Toronto respond inelastically to the 2% in 50 year ground motions in the short period range but return to elastic response after a period of one second.
- Vancouver responds inelastically to both 10% in 50 year and 2% in 50 year ground motions in the zero to two second period range.

Table 4.1 CHBDC/AASHTO Response Modification Factor	
Ductile Substructure Element	R
Wall-type piers, in direction of larger dimension	2.0
Reinforced concrete pile bents	
• Vertical piles	3.0
• Batter piles	2.0
Single columns	
▪ Ductile reinforced concrete	3.0
▪ Ductile steel	3.0
Steel, composite steel and concrete pile bents	
• Vertical pile	5.0
• 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
▪ Nominally ductile steel braces	2.5
*CHBDC only	

Table 4.2 ATC/MCEER Response Modification Factors				
MCE-Substructure Element Response Modification Factor, R_b $R = 1 + (R_b - 1) \frac{T}{1.25T_s} \leq R_b$	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall Piers-larger dimension	2	3	1	1.5
Columns-Single and Multiple	4	6	1.5	2.5
Pile Bents and Drilled Shafts-Vertical Piles above ground	4	6	1.5	2.5
Pile Bents and Drilled Shafts-Vertical Piles-2 diameter below ground level-no owners approval required	1	1.5	1	1
Pile Bents and Drilled Shafts-Vertical Piles-in ground-owners approval	N/A	2.5	N/A	1.5
Pile Bents with Batter Piles	N/A	2	N/A	1.5
Seismically Isolated Structures	1.5	1.5	1	1.5
Steel Braced Frame-Ductile Components	3	4.5	1	1.5
Steel Braced frame-Nominally Ductile Components	1.5	2	1	1
Expected Earthquake-All Elements	1.3	1.3	0.9	0.9

Figure 4.1 Positive Displacement Ductility 10% In 50 Year Short Period Ground Motion

Figure 4.2 Positive Displacement Ductility 10% In 50 Year Long period Ground Motion

Figure 4.3 Positive Displacement Ductility 2% In 50 Year Short Period Ground Motion

Figure 4.4 Positive Displacement Ductility 2% In 50 Year Long Period Ground Motion

Figure 4.5 Montreal/Ottawa Positive Displacement Ductility

Figure 4.6 Toronto Positive Displacement Ductility

Figure 4.7 Vancouver Positive Displacement Ductility

Chapter 5

5. Characteristics of Bridge Structures

5.1 General

Highway bridges are critical links in the Canadian transportation system. In Ontario the 400 series highways form the most important transportation network in the province. Two types of bridges, which are part of this transportation network are chosen for this study. The first is the Barnsdale Road Underpass, which is a two-lane bridge passing over the 416 highway south of Ottawa. The second is the CNR Overhead EBL, which carries highway 417 east bound lanes over the CNR easement in eastern Ottawa. The force-deformation relationship of the bridge columns has been determined using the BIAx program (Wallace, 1992). The mode shapes and frequencies of the structures have been determined using the NEABS program (Penzien et al., 1981), which are discussed in Chapter 6. The information from the moment-curvature analysis and the dynamic characteristics, such as the structural mass, period and yield force are input in the NONSPEC program (Mahin et al., 1983) to determine the displacement ductility. The displacement, rotational and curvature ductility are determined from dynamic time-history analysis using the NEABS program. The analyses have been performed for Montreal, Ottawa, Toronto and Vancouver using ground motions compatible with the 10% in 50 year and 2% in 50 year UHS in the short and long period range. The general characteristics of the bridges are discussed in this Chapter, while the dynamic characteristics and analysis are discussed in Chapter 6.

5.2 Concrete Stress-Strain Relationship

The stress-strain relationship for reinforced concrete adopted in this study is based on the modified Kent and Park model. In 1971 Kent and Park (Kent et al., 1971) proposed the stress-strain relationship for concrete presented in Figure 5.1, with the following properties:

- $\epsilon_{50u} = \frac{3 + 0.002 f'_c}{f'_c - 1000}$
- $p'' = \frac{2(b'' + d'') A'' s}{b'' d'' s}$
- $\epsilon_{50h} = 0.75 p'' \sqrt{\frac{b''}{s}}$
- $\epsilon_{50c} = \epsilon_{50h} + \epsilon_{50u}$

where A'' is the cross sectional area of the transverse reinforcement or spiral, s is the center to center spacing of the hoops and b'' and d'' are the dimensions of the confined concrete core. In 1989 Mander, Priestly and Park (Mander et al., 1989) published a paper proposing the Stress-Strain model presented in Figure 5.2, which uses the following properties (different nomenclature compared to the previous model):

- $f'_{cc} = f'_c \left(2.254 \sqrt{1 + \frac{7.94 f'_c}{f'_c} - \frac{2 f'_l}{f'_c} - 1.254} \right)$
- $f'_l = 0.95 \frac{2 f_{yh} A_{sp}}{D' s}$
- $\epsilon_{cc} = 0.002 \left[1 + 5 \left(\frac{f'_c}{f'_{cc}} - 1 \right) \right]$
- $E_c = 5000 \sqrt{f'_{cc}}$

Where f_{yh} is the yield strength of the spiral, D' is the diameter of the spiral, A_{sp} is the bar area and f'_{co} is the unconfined concrete strength. The BIAx program is used to determine the concrete-stress strain properties of the columns in this study. The BIAx program uses a modified Kent-Park concrete stress-strain relationship as presented in Figure 5.3.

5.3 Column Plastic Hinge Properties

Modern bridge design relies on ductile substructure elements to behave inelastically during seismic excitation. The ductile substructure elements are relied upon to dissipate seismic energy rather than the superstructure because large deformations of the superstructure are undesirable and more costly to repair. Plastic hinges are formed where inelastic actions occur. The plastic hinge region is located at the top and/or bottom of the column where it is connected to the superstructure or foundation. Plastic hinges form because of large moments exceeding the elastic yield level. In the case where the columns are continued below ground as a drilled shaft foundation a plastic hinge may form at the interface of the column and the ground surface. Physically the formation of a plastic hinge within a column begins with the cracking of the concrete, followed by spalling of the concrete cover and eventual fracture of the transverse reinforcement. Fracture of the hoop or spiral transverse reinforcement is taken as failure of the reinforced concrete section because once the concrete becomes unconfined, the section will rapidly lose strength and fail. In the computer model here, the length of the plastic hinge zone is determined by following the recommendations by Calvi and Priestley (Priestly et al., 1996),

$$L_p = 0.08L + 0.022f_{ye} \geq 0.044f_{ye}d_{bl}$$

where f_{ye} is equal to $1.1f_y$, d_{bl} is the diameter of the longitudinal reinforcement and L is the distance from the critical section of the plastic hinge to the point of contraflexure.

The yield displacement can be related to the yield curvature using the following relationship (Priestley et al., 1996),

$$\Delta_y = \frac{\phi_y L^2}{3}$$

where L is the distance from the point of maximum moment to the point of contraflexure.

For single column bents under earthquake excitations, the column deforms in single curvature in the transverse direction and double curvature in the longitudinal direction.

For a multiple column bent, the columns are expected to behave in double curvature in both the transverse and longitudinal directions. It is assumed for both single and multiple column bents that the connections of the substructure to the superstructure and foundation are rigid. In single curvature, the column is likely to develop one plastic hinge zone at the base of the column, whereas in double curvature, two plastic hinge zones are commonly observed from bridge performance in previous earthquakes, one at the base and one at the top of the column, as presented in Figures 5.4 and 5.5 (CALTRANS, 2004).

5.4 Barnsdale Road Underpass

Barnsdale road Underpass is a two span, single column, prestressed bridge. The bridge was built in 1990. The bridge was likely designed using the Ontario Highway Bridge Design Code specifications (1979/1991) or AASHTO (1983/1991) bridge design specifications. The column is monolithic to the bridge deck and has laminated elastomeric bearings and expansion joints at each abutment. The bridge has a piled foundation. The total length of the bridge including abutments is 75 meters, the total length of the bridge

from the center line of the abutment bearings is 64 meters. The height of the column from the top of the pile cap to the deck is 7.28 meters. An elevation view of the Barnsdale Road underpass is presented in Figure 5.6. An elevation and cross section view of the column are presented in Figure 5.7. The column is 1500 mm in diameter with 30- #35 longitudinal bars and a #15 spiral at 50 mm pitch. The concrete cover is 80 +/-20 mm. A cross section view of the deck is presented in Figure 5.8. The deck is 9460 mm wide and 1250 mm deep at the center. Between the abutments and the column a hollow concrete section is used, while at the abutments and over the column the section is solid. The modified Kent and Park model is used to model the stress-strain behaviour of the concrete (Wallace, 1992). The properties of the reinforced concrete used in the analysis are presented in Table 5.1. The member cross section properties are presented in Table 5.3. The column interaction diagram and moment-curvature relationship are presented in Figures 5.9 and 5.10. A bilinear approximation is used to determine the yield curvature (Priestly et al., 1996), the resulting curvature is found to be ϕ_y (/m)=0.00215.

5.5 CNR Overhead EBL

The CNR Overpass is a five span, multi-column, prestressed girder bridge. The bridge was built in 1972. The bridge was likely designed using AASHTO bridge design specifications. An elevation view of the bridge is presented in Figure 5.11. The east bound lanes are chosen for the study here because they are a wider and taller structure than the west bound lanes although the overall design of the structures is the same. The CNR Overpass has a 43 degree skew, which is neglected for simplicity in the analysis. The bridge connections to the abutments and superstructure are supported on bearings.

There are expansion joints at the abutments and the two exterior piers, while the two central piers are fixed to the superstructure. The bridge has a piled foundation. The total length of the bridge including abutments is 94 meters, the total length of the bridge from the center line of the abutment bearings is 72 meters. The two exterior spans are 11.2 meters in length, and the three interior spans are 16.8 meters in length (from the center line of the abutment bearings). The bridge superstructure is constructed using seven AASHTO II prestressed girders, continuous over the superstructure. The width of the bridge varies from 14 meters to 18 meters along its length from north to south. At the abutments and piers the girders are rigidly connected to each other using diaphragms. The bridge is supported on four multi-column bents and each bent has three columns. The columns are rigidly connected to a pier cap. The columns at Pier #1 and #4 adjacent the north and south abutments are approximately 4.25 meters in length. At pier #1 and #4 the bridge superstructure is supported on the pier cap by elastomeric bearings. The columns at Pier #2 and #3 are approximately 8 meters in length. At pier #2 and #3 the bridge superstructure is supported on the pier cap by elastomeric bearings, but the pier cap and superstructure are rigidly connected using dowels. The columns of pier #2 and #3 are connected by a collision strut, which is located approximately 3.6 meters above the pile cap. An elevation and cross section view of piers #2 and #3 is presented in Figure 5.12. The pier columns are 914 mm in diameter with 12- 35mm longitudinal bars and a 15mm spiral at 50 mm pitch. The concrete cover is 76 mm. The modified Kent and Park model is used to model the stress-strain behaviour of the concrete (Wallace, 1992). The material properties of the reinforced concrete members in the example structures considered in the analysis are presented in Table 5.2. The member cross section

properties are presented in Table 5.4. The column interaction diagram and moment-curvature relationship are presented in Figures 5.13 and 5.14. A bilinear approximation is used to determine the yield curvature (Priestly et al., 1996), the resulting curvature is found to be ϕ_y (/m)=0.004.

Table 5.1 Barnsdale Road Underpass Reinforced Concrete Properties	
f_c	35 MPa
ϵ_o (unconfined)	0.002
f_t	3 MPa
F'_{cc}	49.5 MPa
ϵ_{cc}	0.006
E_c	28000 MPa
ν	0.2
γ	2400 kg/m ³

Table 5.2 CNR Overhead EBL Reinforced Concrete Properties	
f_c	27.5 MPa
ϵ_o (unconfined)	0.002
f_t	2.6 MPa
F'_{cc}	49.5 MPa
ϵ_{cc}	0.01
E_c	26000 MPa
ν	0.2
γ	2400 kg/m ³

Table 5.3 Barnsdale Road Overpass				
Element type	area m ²	inertia x m ⁴	inertia y m ⁴	inertia z m ⁴
Deck Section 1	8.378	3.549	1.492	19 872
Deck Section 2	10.852	3.768	1.316	28.39
Column Section	1.767	0.754	0.377	0.377

Table 5.4 CNR Overhead EBL				
Element type	area m ²	inertia x m ⁴	inertia y m ⁴	inertia z m ⁴
Deck Section 1	4.695	66.6	0.554	65.5
Deck Section 2	15.706	268.6	1.637	267
Pier cap	1.28	0.228	0.11	0.169
collision strut	0.348	0.015	0.017	0.006
column section	0.656	0.069	0.034	0.034

Figure 5.1 Kent And Park Stress-Strain Relationship

Figure 5.1: Stress-Strain for Unconfined or Confined Concrete, (Park et al., 1971).

Figure 5.2 Mander, Priestly And Park Stress-Strain Relationship

Figure 5.2: Stress-Strain Model for Monotonic Loading of Confined and Unconfined Concrete, (Mander et al., 1989).

Figure 5.3 Biax Modified Kent And Park Stress-Strain Relationship

Figure 5.3: Concrete Stress-Strain Relations, (Wallace, 1992).

Figure 5.4 Column Displacement Single Curvature

Figure 5.4: Local Displacement Capacity-Cantilever w/ Fixed Base, (CALTRANS, 2004).

Figure 5.5 Column Displacement Double Curvature

Figure 5.5: Local Displacement Capacity-Framed Column, Assumed as Fixed-Fixed, (CALTRANS,2004).

Figure 5.6 Elevation View Barnsdale Road Overpass

Figure 5.7 Elevation And Cross Section View Barnsdale Road Overpass Column

Plan view of a bridge structure. The bridge is symmetrical about a central vertical axis. The total width is 9460. The total length is 1115. The bridge has a central span of 2730 and two side spans of 2000. The bridge is supported by three piers. The bridge deck is 600 wide. The bridge has a 2% slope. The bridge is located on Barnsdale Rd. The bridge is shown in plan view with dimensions in feet. The bridge is symmetrical about a central vertical axis. The total width is 9460. The total length is 1115. The bridge has a central span of 2730 and two side spans of 2000. The bridge is supported by three piers. The bridge deck is 600 wide. The bridge has a 2% slope. The bridge is located on Barnsdale Rd. The bridge is shown in plan view with dimensions in feet.

Figure 5.9 Barnsdale Road Underpass Interaction Diagram

Figure 5.10 Moment Curvature-Barnsdale Rd. Overpass

Figure 5.11 Elevation View CNR Overhead EBL

Figure 5.12 Elevation And Cross Section View CNR Overhead Column

Figure 5.13 Interaction Diagram-CNR Overpass

Figure 5.14 Moment Curvature-CNR Overpass

Chapter 6

6. Dynamic Time History Analysis

6.1 General

The impact and significance of recent seismic design developments and adoption of the UHS format of seismic hazard specification in the 2005 NBCC on the bridge design code practices in Canada are evaluated by means of dynamic time history analysis. The purpose of this analysis is to obtain insights in light of the recent developments in earthquake engineering design that will lead to better performance objectives and use the latest scientific and engineering knowledge to perform safer and more efficient seismic design of highway bridges in Canada.

Three dimensional structural models of the Barnsdale Road Underpass Road and CNR Overpass bridges have been established by the NEABS program (Penzien et al., 1981). The mode shapes and frequencies of the structures are evaluated. The mass and yield displacement of the Barnsdale Road Underpass system obtained from the moment-curvature analysis and the fundamental period obtained from the NEABS analysis are input into the NONSPEC program to determine the system specific response to the input ground motions for each city, at specific probability levels. Non-linear time history analysis have been performed on the Barnsdale Road Underpass using the NEABS program to determine the response of the structures to the UHS compatible ground motions. The NONSPEC displacement time-history responses are compared to the NEABS displacement time-history response for the Barnsdale Road Underpass. A non-linear time history analysis is performed of the CNR Overpass using the NEABS program. The 2% in 50 year UHS spectra for soil types A to E for Montreal/Ottawa,

Toronto and Vancouver have been constructed by multiplying the UHS for the reference ground condition by the appropriate F_a and F_v factors of NBCC 2005. Ground motions compatible with the short and long period UHS are used to compare the effect of the 2005 NBCC soil factors on the seismic response of Barnsdale Road Underpass. The seismic response of the Barnsdale Road Underpass and the CNR Overpass has been evaluated using displacement and rotation requirements of ATC/MCEER 2003.

6.2 Analysis Techniques

The frequencies and mode shapes of the structures are determined using the NEABS program developed by previous graduate students at Carleton University. This version of the NEABS program utilizes the Lanczos method (Phung, 2005), which is an extension of the Rayleigh-Ritz method to find the dynamic properties of the system. In the Rayleigh-Ritz method an assumed deflected shape is used as the starting vector for the frequency analysis. In the Lanczos method the assumed deflected shape corresponds to the static application of the dynamic load. Once the dynamic properties of the system have been calculated, the Rayleigh Damping coefficients are found using the first and third mode frequencies. Rayleigh damping assumes damping is proportional to the mass and stiffness matrices as described in the following equations (Clough et al., 1993):

$$C = \alpha M + \beta K$$

$$\begin{Bmatrix} \xi_1 \\ \xi_3 \end{Bmatrix} = \frac{1}{2} \begin{bmatrix} 1/\omega_1 & \omega_1 \\ 1/\omega_3 & \omega_3 \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix}$$

The first and third mode damping ratios are assumed to be 5% of critical, 0.05, and the equations have been solved for both Barnsdale Road Underpass and the CNR overhead. The dynamic response is determined in the NEABS program using the constant acceleration integration technique.

The node placement for the Barnsdale Road Underpass and CNR Overpass are presented in Figures 6.1, 6.2 and 6.3. The dynamic properties of the bridges are presented in Table 6.1 to 6.4.

6.2.1 Yield Displacement and Rotation

A summary of the yield displacement used in the analysis is presented in Table 6.5. The yield displacement is calculated using the formula (Priestley et al., 1996):

$$\Delta_y = \frac{\phi_y l^2}{3}$$

with the assumption of either single or double curvature as shown in the table, where ϕ_y is found from the moment curvature relationship defined in Chapter 5. Displacement capacity verification (push-over) analysis to determine the yield displacement is performed by recording the top of column displacement at the first yield of any column member. The yield displacement found by the push-over analysis result are selected to calculate the displacement ductility factor. In all cases, the push-over analysis results lead to a lower yield displacement value and therefore higher displacement ductility demand. The rotational ductility factors have been calculated using the yield curvature from the moment-curvature analysis multiplied by the length of the section under consideration. The maximum rotational ductility demand is found by dividing the maximum rotation by the yield rotation as follows,

$$\theta_y = \phi_y l_p$$

$$\mu_\theta = \frac{\theta_{\max}}{\theta_y},$$

where l_p is the plastic hinge length or the length of the column section under consideration.

6.2.2 Recommended Response Modification Factors

The displacement and rotational ductility demand are related to the force reduction factor or the base response modification factor (Priestley et al., 1996), as stated in Chapter 5. The recommended R factor in CHBDC 2000 is 3.0 for single columns and the value is 5.0 for multiple columns bents. The recommended base response modification factor used with seismic design and analysis procedure E in ATC/MCEER 2003 is 6.0 for single and multiple columns. CALTRANS seismic design criteria 2004 recommend ductility demands less than or equal to 4.0 for single columns bents on fixed foundation and 5.0 for multi-column bents on fixed or pinned foundations. The displacement and rotational ductility demand calculated are compared to 6.0, as recommended by ATC/MCEER 2003 because the analysis procedure used in this study is similar to seismic design and analysis procedure E recommended by ATC/MCEER. The base response modification factor, R for Montreal/Ottawa, Toronto and Vancouver soil types A to E, 2% in 50 year UHS have been calculated using the formula,

$$R = 1 + (R_b - 1) \left(\frac{T}{1.25T_s} \right) \leq R_b,$$

where R_b is equal to 6.0 and $T_s = S_{dl}/S_{ds}$ and $S_{dl} = F_v S_l$ and $S_{ds} = F_a S_s$. The calculated value of R is greater than 6.0 for all cases, except Vancouver soil type E, where the value of R is 4.29.

6.3 Dynamic Analysis

6.3.1 Barnsdale Road Underpass

Dynamic time history analysis are performed on Barnsdale Road underpass using the 2% in 50 year and 10% in 50 year short and long period compatible ground motions for Montreal, Ottawa, Toronto and Vancouver in the longitudinal and transverse directions. The results of the longitudinal analyses are compared to similar analyses, which are conducted using the NONSPEC program. The NONSPEC program calculates the response of a viscously damped, inelastic single degree of freedom system, as presented in Figure 6.4 (Mahin et al., 1983). The individual mode shapes of a bridge structure behave as generalized single degree of freedom systems. It is common for many typical bridge structures to have their response to seismic excitation dominated by one primary mode. As can be seen in Table 6.2, the Barnsdale Road underpass responds primarily in the first mode to seismic excitation in the longitudinal (x) direction. Therefore it is possible to compare the longitudinal response of the simple bridge structure to a single degree of freedom system. Figures 6.5 to 6.16 compare the longitudinal displacement time history response at the top of the single column in the NEABS program to the NONSPEC program and the input accelerogram as a reference. The Figures indicate that the shape and magnitude of the column displacement time history response of the Barnsdale Road Underpass is similar to that of the single degree of freedom system model used in the NONSPEC program in the longitudinal direction. The percent difference between the maximum column displacement responses calculated using NONSPEC and NEABS are compared in Table 6.6, using the NEABS displacement results as a reference value. The magnitude of the NONSPEC value is generally larger

than the NEABS value, with the exception of Montreal/Ottawa and Vancouver 2% in 50 year short period. A summary of the maximum and minimum displacements and the positive and negative displacement ductility for the NONSPEC and NEABS analysis are shown in Table 6.7. The NEABS values are considered more accurate than the NONSPEC values because the NEABS analysis calculates the response of the structure as a whole whereas the NONSPEC program assumes a single degree freedom response (Mahin et al., 1983).

From examination of Figures 6.5 to 6.16 it is noted that the Barnsdale Road Underpass responds non-linearly to the input ground motions in the longitudinal direction matching the Montreal/Ottawa 2% in 50 year short and long period UHS and the Vancouver 10% in 50 year and 2% in 50 year short and long period UHS. The response is linear for all cases for time history records matching the Toronto UHS. The maximum displacement ductility demands calculated in Table 6.7 also indicate non-linear behaviour for the previously mentioned ground motions. In Montreal/Ottawa the maximum displacement ductility demand is found to be 1.75 for the 2% in 50 year short period compatible ground motions and 4.3 for the 2% in 50 year long period compatible ground motions. In Vancouver the maximum displacement ductility demand for the 10% in 50 year ground motions is 1.3 for both short and long period cases. While for the 2% in 50 year compatible short and long period ground motions the maximum displacement ductility demand is found to be 6.9 and 3.3 respectively. The large value found for the Montreal/Ottawa 2% in 50 year long period ground motion is the result of the large residual displacement. The calculated value of $\mu = 6.9$ is greater than the ATC/MCEER 2003 recommended value of 4.29.

The maximum rotation and the maximum rotational ductility along the length of the column have been found. The column maximum rotation and calculated rotational ductility demand are presented in Table 6.9. The maximum rotational ductility demands for Montreal/Ottawa 2% in 50 year long period ground motion and Vancouver 2% in 50 year short and long period ground motions are greater than one, which indicate non-linear behaviour. The maximum rotational ductility value for Montreal/Ottawa is 1.7 and for Vancouver short and long period the maximum rotational ductility value is 3.1 and 1.1 respectively. The maximum displacement and rotation response in the transverse direction are presented in Table 6.8 and 6.9. The response in the transverse direction is considerably smaller than the longitudinal direction and the displacement and rotational ductility demands are less than 1.0.

6.3.2 CNR Overhead EBL

Dynamic Time history analysis using the 2% in 50 year and the 10% in 50 year short and long period UHS compatible ground motions have been performed for the CNR overhead bridge EBL in the longitudinal and transverse directions. The response displacement at the top of the central column of pier 2 in the longitudinal direction is presented in Figures 6.17 to 6.28. The time history displacement is plotted with the time history accelerogram as a reference.

The longitudinal displacement time history response to the Montreal/Ottawa 2% in 50 year short and long period ground motions indicate non-linear behaviour. The longitudinal displacement time history response to the Vancouver 10% in 50 year and 2% in 50 year short and long period ground motions indicate non-linear behaviour. Whereas the longitudinal displacement time history response to the Toronto 10% in 50 year and

2% in 50 year short and long period ground motions as well as the Montreal/Ottawa 10% in 50 year short and long period ground motions indicate linear response.

The column maximum displacement and displacement ductility in the longitudinal and transverse directions are presented in Table 6.8. The column maximum rotation and rotational ductility in the longitudinal and transverse directions are presented in Table 6.9. Like the Barnsdale Road Underpass, the CNR overhead bridge has the maximum displacement and rotation response to input ground motion in the longitudinal direction. In the longitudinal direction maximum displacement ductility demands greater than 1.0 are observed for the Montreal/Ottawa 2% in 50 year long period ground motion, which had a displacement ductility demand of 1.5. In the longitudinal direction maximum displacement ductility demands greater than 1.0 are observed for the Vancouver 2% in 50 year short and long period ground motions, with values of 2.8 and 1.5 respectively.

In the longitudinal direction maximum rotational displacement ductility demands greater than 1.0 are observed for the Montreal/Ottawa 2% in 50 year short and long period ground motions with values of 5.1 and 1.7 respectively. The Vancouver 10% in 50 year long period ground motion has a rotational ductility demand of 1.1. The 2% in 50 year short and long period ground motions in Vancouver have rotational ductility demand values of 2.3 and 1.2 respectively. The response in the transverse direction is considerably smaller than the longitudinal direction and the displacement and rotational ductility demands are less than 1.0.

6.4 Dynamic Analysis using Site Class A to E

Ground motions matching the 2% in 50 year short and long period UHS for Site Class A, B, C, D and E in Montreal/Ottawa, Toronto and Vancouver were found using the

minimum variance criterion. The minimum variance for each time history and scaling factor combination is used to select the ground motion that best fits the target UHS (Phung, 2005) in the short and long period range, as described in Chapter 3.

. The magnitude, distance, scale factor, duration and maximum acceleration (unscaled) for each ground motion and city are presented in Tables 6.10, 6.11 and 6.12. To best fit the UHS data, a larger array of scale factors is used from 0.1 to 3.0 in increments of 0.1. The UHS for NBCC site class A to E and the seismic response coefficient of CHBDC soil type 1 and 4 corresponding to 10% in 50 year PGA are compared for Montreal/Ottawa, Toronto and Vancouver in Figures 6.29, 6.30 and 6.31.

In the short period range the 2% in 50 year UHS value are greater than CHBDC spectral values. In the intermediate range however the UHS values decrease more rapidly than the corresponding CHBDC values. By the one second period the 2000 CHBDC soil type 4 values are larger than the corresponding 2005 NBCC Soil type D and E factors. The 2% in 50 year UHS for the 2005 NBCC soil types A and B are less than the corresponding 2000 CHBDC values by the 0.2 second in Montreal/Ottawa, the 0.3 second period in Toronto and the 0.6 second period in Vancouver. The reference ground condition values for the 2% in 50 year UHS (Soil Type C) and 2000 CHBDC (Soil Type 1) have been compared in Chapter 3. The 2005 NBCC soil factors have the greatest impact on the UHS in the region of lowest seismicity, Toronto. However in the short period range the UHS values for soil type D and E are significantly higher than CHBDC values in all the cities.

Dynamic time history analysis have been performed on Barnsdale Road Underpass using ground motions compatible with the 2% in 50 year short and long period UHS

matching soil type A to E for Montreal/Ottawa, Toronto and Vancouver. The input ground motions are applied in the longitudinal direction because in the previous analysis it was found that the longitudinal response is greater than the transverse direction response. The maximum displacement and displacement ductility at the top of the single column are presented in Table 6.13. The displacement ductility demand for the Ottawa long period ground motion soil type D and E are 19.8 and 25.7. These values are three and four times the maximum R factor of 6 for single column bents recommended by ATC/MCEER. The displacement ductility demand for the Toronto long period ground motion soil type E is 17.6, which is approximately three times the recommended value. In Vancouver the displacement ductility demands for soil type A are 1.6 and 1.7 but the values for soil type B, C, D and E range from 3.3 to 8.9. Displacement ductility demands greater than 6 or greater than 4.29 for Vancouver soil type E, indicate ductility demands, which exceed the 2003 ATC/MCEER recommended base response modification factor.

The rotation and rotational ductility demands are presented in Table 6.14. The rotational ductility demand for the Ottawa long period ground motion soil type D and E are 10.3 and 10.1, which are greater than the values recommended by ATC/MEER. The rotational ductility demand for the Toronto long period ground motion soil type E is 5.6. In Vancouver the rotational ductility demand ranges from 0.75 to 3.6.

6.5 Design Displacement and Rotation

The seat width and plastic rotation capacity of the Barnsdale Road Underpass and the CNR Overpass are calculated using ATC/MCEER 2003 recommended procedures. The calculated values are compared to the time history values. The calculated required seat width is compared to the maximum time history displacement response and the

calculated plastic rotation capacity is compared to the maximum time history rotation response.

6.5.1 Plastic Hinge Length

The plastic hinge length is calculated because it is required in the ATC/MCEER 2003 procedure to determine the plastic rotation capacity. The method used to determine the plastic hinge length by ATC/MCEER differs somewhat from the procedure described in Chapter 5. The ATC/MCEER 2003 method has been adopted in this study to determine the plastic hinge length. ATC/MCEER recommends the following criteria for determining the maximum length of the plastic hinge zone (ATC/MCEER, 2003):

1. 450mm
2. for steel columns 1/8 the clear height of the column
3. for reinforced concrete 1/6 the clear height of the column, plus
 - $D(\cot \theta + 1/2 \tan \theta)$
 - $1.5(0.08M/V + 4400\varepsilon_y d_b)$
 - $M/V(1-M_y/M_{p0})$

Where

D=transverse column dimension in the direction of bending

$$\theta = \text{principal crack angle, } \tan \theta = \left(\frac{1.6\rho_v A_v}{\Lambda\rho_l A_g} \right)^{0.25}$$

ε_y =yield strain of longitudinal reinforcement

d_b =longitudinal bar diameter

M=maximum column moment

V=maximum column shear

M_y =column yield moment

M_{po} =column plastic overstrength moment

Λ = fixity factor (1 for fixed-free, 2 for fixed-fixed)

A_v =shear area of concrete $0.8A_g$

A_g =gross area of column

ρ_l =ratio of longitudinal reinforcement area to gross area of section

ρ_v =ratio of transverse reinforcement $\frac{2A_{BH}}{sD''}$

s =pitch of spiral

D'' =center to center diameter of spiral

A_{bh} =area of spiral

The maximum plastic hinge length is determined for Barnsdale Road Underpass and the CNR Overpass. The maximum value is found to correspond to the one sixth the column length criteria, which is 1.21 meters for Barnsdale Road Underpass and 1.33 for the CNR Overpass.

6.5.2 Minimum Seat Width

Loss of span due to inadequate seat width has caused bridge failures in many previous earthquakes (Priestley et al., 1996). Therefore the maximum displacement found from the time history analysis is compared to ATC/MCEER seat width requirements.

ATC/MCEER recommends the following criteria for determining minimum seat width required (ATC/MCEER, 2003):

- The minimum seat width will be greater than 1.5 times the displacement of the superstructure.

- Or $N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} * \sqrt{1 + \left(2\frac{B}{L} \right)^2} \right] \frac{(1 + 1.25F_v S_l)}{\cos \alpha}$

L=the distance between joints in meters

H=the tallest pier between the joints in meters

B=the width of the superstructure in meters

α =the skew angle

Where B/L is less than or equal to 3/8.

The minimum seat width is calculated for Barnsdale Road Underpass and the CNR Overpass. The calculated seat width and the maximum displacement times 1.5 are presented in Table 6.15 and 6.16 for Barnsdale Road and the CNR Overhead respectively.

6.5.3 Plastic Rotation Capacities

ATC/MCEER recommends the plastic rotational capacity of plastic hinges for the life safety performance objective as follows (ATC/MCEER, 2003):

$$\theta_p = 0.11 \frac{L_p}{D} (N_f)^{-0.5} \text{ radians,}$$

Where N_f is the number of cycles of loading expected at the maximum displacement amplitude,

$$N_f = 3.5(T_n)^{-1/3}, \text{ for } 2 < N_f < 10$$

Where T_n is the fundamental period of the structure.

Alternatively a conservative value of $\theta_p = 0.035$ may be used. For the operational performance objective a value of $\theta_p = 0.01$ may be used.

The plastic rotational capacity of the Barnsdale Road Underpass and the CNR Overpass has been calculated. The value of plastic rotation calculated using the above formula is added to the yield rotation, which has been determined from the moment-curvature relation. The total rotational capacity is found by adding the plastic rotation to the yield rotation. The total rotational capacity and the maximum rotation found from the time history analysis for Barnsdale Road Underpass and the CNR Overpass are shown in Table 6.15 and 6.16 respectively.

6.6 Summary

The dynamic properties of the Barnsdale Road Underpass and CNR Overpass are found using the NEABS program. The longitudinal displacement found using the NONSPEC program has been found to be within an acceptable range of the results of the NEABS program for most load cases. The seismic response in the transverse direction is less than that in the longitudinal direction for both structures and all load cases. The longitudinal response of the Barnsdale Road Underpass and CNR Overpass have been observed to be non-linear for the Montreal/Ottawa 2% in 50 year short and long period ground motions and the Vancouver 10% in 50 year and 2% in 50 year short and long period ground motions.

ATC/MCEER 2003 recommend a maximum base response modification factor, $R=6.0$ for single and multiple column bents in conjunction with seismic design and analysis procedure E. The maximum base response modification factor have been calculated for each ground motion and are found to equal 6.0 except for the case of Vancouver soil type E where $R=4.29$. The calculated displacement and rotational ductility values are under 6 for all cases except the Vancouver 2% in 50 year short period

compatible ground motion, which has been found to be 6.9. The largest response is the Montreal/Ottawa 2% in 50 year long period ground motion and the Vancouver 2% in 50 year short and long period ground motion.

The required seat width has been calculated and compared to the maximum response displacement times 1.5. The calculated required seat width ranged from 426mm to 578mm and is larger than 1.5 times the maximum response displacement in all cases. The seat width provided is 900mm for Barnsdale Road Underpass and 685mm for the CNR Overpass.

ATC/MCEER 2003 guidelines set limits on the plastic rotation capacity of columns. The total allowable rotational capacity should be the minimum of the calculated value or 0.035 plus the yield rotation. The maximum column rotation is found to be less than the minimum value for all cases.

The maximum displacement and rotation response of the Barnsdale Road Underpass to the 2% in 50 year UHS for soil types A to E in Montreal/Ottawa, Toronto and Vancouver have been found. The displacement ductility demand for the Ottawa long period ground motions for soil type D and E and for Vancouver short period soil type B, C, D and E and the long period soil type B and E are found to be greater than the recommended value. The rotational ductility demand for the Ottawa long period ground motions for soil type D and E have been found to be greater than 6. In the case of Montreal/Ottawa and Toronto the increase in the 2% in 50 year UHS for soil types D and E creates response demands, which are greater than the recommended values. In the case of Vancouver the 2% in 50 year UHS for soil types B, D, D and E create response demands which exceed the recommended values.

Table 6.1 Barnsdale Road Overpass Frequencies		
Mode #	frequency (rad/sec)	period (sec)
1	10.4994	0.5984
2	12.9890	0.4837
3	13.0919	0.4799
4	25.0392	0.2509
5	30.5943	0.2054
6	53.2235	0.1181
7	74.7502	0.0841
8	82.7338	0.0759
9	151.4643	0.0415
10	155.4925	0.0404

Table 6.2 Barnsdale Road Overpass Modal Mass Participation								
x direction			y direction			z direction		
mode #	modal %	total %	mode #	modal %	total %	mode #	modal %	total %
1	91.87	91.87	4	67.26	67.26	8	90.69	90.69
2	7.17	99.04	9	12.58	79.83	7	7.86	98.55
3	0.21	99.25	20	6.5	86.34	16	0.33	98.87
11	0.17	99.42	11	4.52	90.86	15	0.02	98.89
18	0.12	99.54	19	2.41	93.27	17	0	98.89
19	0.07	99.61	15	1.55	94.82	9	0	98.89
12	0.07	99.67	10	0.94	95.75	6	0	98.89
9	0.06	99.73	6	0.81	96.56	10	0	98.89
5	0.04	99.77	3	0.52	97.08	14	0	98.89
20	0.03	99.81	13	0.52	97.6	18	0	98.89
6	0.03	99.84	5	0.43	98.03	11	0	98.89
7	0.03	99.86	17	0.11	98.14	13	0	98.89
17	0.01	99.87	14	0.1	98.24	5	0	98.89
14	0.01	99.88	12	0.08	98.32	12	0	98.89
10	0.01	99.89	16	0.07	98.39	19	0	98.89
15	0	99.89	18	0.01	98.4	4	0	98.89
13	0	99.89	7	0	98.4	20	0	98.89
8	0	99.9	2	0	98.4	3	0	98.89
4	0	99.9	8	0	98.4	2	0	98.89
16	0	99.9	1	0	98.4	1	0	98.89

Table 6.3 CNR Overhead EBL		
mode no.	frequency (rad/sec)	period (sec)
1	12.6642	0.4961
2	14.8394	0.4234
3	19.8882	0.3159
4	29.9010	0.2101
5	39.2938	0.1599
6	45.3142	0.1387
7	46.9993	0.1337
8	47.5371	0.1322
9	47.8589	0.1313
10	56.2175	0.1118

Table 6.4 CNR Overhead EBL Modal Mass Participation								
x direction			y direction			z direction		
mode #	modal %	total %	mode #	modal %	total %	mode #	modal %	total %
4	54.26	54.26	22	25.08	25.08	1	60.55	60.55
5	37.96	92.22	23	14.67	39.76	2	20.4	80.94
39	1.14	93.36	21	11.96	51.72	3	10.71	91.65
3	0.72	94.09	38	6.14	57.86	6	2.21	93.86
30	0.47	94.55	26	4.94	62.8	10	0.68	94.55
46	0.4	94.96	20	3.66	66.45	9	0.68	95.23
40	0.34	95.3	42	2.17	68.62	7	0.6	95.82
47	0.32	95.62	40	1.85	70.48	4	0.54	96.37
6	0.3	95.92	43	1.8	72.27	11	0.5	96.86
14	0.22	96.14	25	1.44	73.71	12	0.46	97.32
24	0.18	96.32	90	1.37	75.08	8	0.39	97.71
13	0.17	96.49	36	1.25	76.33	5	0.33	98.04
55	0.17	96.66	45	1.09	77.42	25	0.23	98.28
44	0.16	96.82	82	0.96	78.38	29	0.18	98.46
48	0.14	96.96	52	0.89	79.26	28	0.16	98.62
57	0.1	97.06	34	0.87	80.13	32	0.12	98.74
37	0.1	97.15	54	0.85	80.98	36	0.09	98.83
49	0.09	97.25	35	0.68	81.66	20	0.08	98.92
51	0.08	97.33	89	0.65	82.31	27	0.08	99
29	0.08	97.4	50	0.55	82.86	31	0.07	99.07

Table 6.5 Yield Displacement values used in Ductility Analysis				
	Barnsdale Rd.		CNR Overhead	
ϕ_y (/m)	0.00215	0.00215	0.004	0.004
Direction	Longitudinal	Transverse	Longitudinal	Transverse
Curvature	Double	Single	Double	Double
Length (m)	7.28	7.28	8	8
Δy (m)	0.0190	0.0380	0.0427	0.0427
Δy Pushover (m)	0.0079	0.0114	0.0373	0.0124

Table 6.6 Percent Difference of the maximum displacement response of the NONSPEC program w.r.t. the NEABS program in the longitudinal direction		
	Positive Displacement Difference (%)	Negative Displacement Difference (%)
Ott/Montreal 500 Short Period	18.8954	9.2264
Ott/Montreal 500 Long Period	11.9617	7.3225
Ott/Mon 2500 Short Period	-41.0701	131.9954
Ott/Mon 2500 Long Period	51.2010	0.1168
Toronto 500 Short Period	15.8879	2.9679
Toronto 500 Long Period	4.4821	10.5061
Toronto 2500 Short Period	52.2596	74.8749
Toronto 2500 Long Period	18.8821	79.8519
Vancouver 500 Short Period	-5.0926	103.9148
Vancouver 500 Long Period	46.4962	28.6275
Vancouver 2500 Short Period	40.4505	-57.3680
Vancouver 2500 Long Period	2.3041	8.4636

Table 6.7 Comparison of Barnsdale Road Overpass Column Displacement and Displacement Ductility, Longitudinal Direction

	NONSPEC				NEABS			
	MAX (m)	MIN(m)	μ	μ	MAX (m)	MIN (m)	μ	μ
Ott/Montreal 500 <0.5s	0.0047	-0.0043	0.5886	-0.5380	0.0039	-0.0039	0.4951	-0.4925
Ott/Montreal 500 >0.5s	0.0047	-0.0038	0.5924	-0.4861	0.0042	-0.0036	0.5291	-0.4529
Ott/Mon 2500 <0.5s	0.0082	-0.0160	1.0316	-2.0266	0.0138	-0.0069	1.7506	-0.8735
Ott/Mon 2500 >0.5s	0.0147	-0.0343	1.8646	-4.3405	0.0097	-0.0343	1.2332	-4.3354
Toronto 500 <0.5s	0.0019	-0.0017	0.2354	-0.2152	0.0016	-0.0017	0.2032	-0.2090
Toronto 500 >0.5s	0.0035	-0.0035	0.4367	-0.4367	0.0033	-0.0031	0.4180	-0.3952
Toronto 2500 <0.5s	0.0066	-0.0066	0.8316	-0.8405	0.0043	-0.0038	0.5462	-0.4806
Toronto 2500 >0.5s	0.0086	-0.0121	1.0823	-1.5367	0.0072	-0.0068	0.9104	-0.8544
Vancouver 500 <0.5s	0.0103	-0.0202	1.2975	-2.5582	0.0108	-0.0099	1.3671	-1.2546
Vancouver 500 >0.5s	0.0155	-0.0118	1.9582	-1.4924	0.0106	-0.0092	1.3367	-1.1603
Vancouver 2500 <0.5s	0.0455	-0.0233	2.1676	-1.1076	0.0324	-0.0546	1.5433	-6.9063
Vancouver 2500 >0.5s	0.0133	-0.0285	1.6861	-3.6013	0.0130	-0.0262	1.6481	-3.3203

Table 6.8 Column Maximum Displacement and Displacement Ductility

	Barnsdale				CNR			
	Longitudinal		Transverse		Longitudinal		Transverse	
	Δ	μ	Δ	μ	Δ	μ	Δ	μ
Mon/Ottawa 500 <0.5s	-0.0039	0.4951	0.0016	0.1388	0.0075	0.1999	-0.0009	-0.0705
Mon/Ottawa 500 >0.5s	0.0042	0.5291	-0.0009	-0.079	0.0106	-0.2845	0.0006	0.0444
Mon/Ottawa 2500 <0.5s	0.0138	1.7506	0.0071	0.6261	-0.022	-0.5965	-0.0037	-0.302
Mon/Ottawa 2500 >0.5s	-0.0342	-4.3354	-0.0076	-0.667	-0.055	-1.4802	0.0039	0.3136
Toronto 500 <0.5s	-0.0017	-0.209	-0.001	-0.0853	0.003	0.08	-0.0003	-0.0282
Toronto 500 >0.5s	0.0033	0.418	-0.0006	0.0551	-0.004	-0.1099	-0.0004	-0.0297
Toronto 2500 <0.5s	0.0043	0.5462	0.0027	0.2345	-0.009	-0.2476	-0.0015	-0.1208
Toronto 2500 >0.5s	0.0072	0.9104	0.0027	0.2389	0.0132	0.355	0.002	0.1598
Vancouver 500 <0.5s	0.0108	1.3671	-0.0055	-0.481	0.0294	0.7882	-0.0033	-0.2634
Vancouver 500 >0.5s	0.0106	1.3367	0.0031	0.274	-0.034	-0.9164	-0.002	-0.1615
Vancouver 2500 <0.5s	-0.0546	-6.9063	-0.0091	-0.7988	-0.105	-2.8311	0.005	0.4024
Vancouver 2500 >0.5s	-0.0262	-3.3203	-0.0053	-0.4645	0.0554	1.4842	0.0035	0.279

Table 6.9 Column Maximum Rotation and Rotational Ductility								
	Barnsdale				CNR			
	Longitudinal		Transverse		Longitudinal		Transverse	
	θ	μ	θ	μ	θ	μ	θ	μ
Mon/Ottawa 500 <0.5s	0.00071	0.17928	0.00027	0.08733	-0.0013	-0.1553	-0.0001	0.0183
Mon/Ottawa 500 >0.5s	0.00064	0.13377	-0.00016	-0.04993	-0.0018	-0.2203	-0.0001	0.0119
Mon/Ottawa 2500 <0.5s	-0.0012	-0.38555	-0.0012	-0.38555	0.0032	0.5459	-0.0005	-0.0832
Mon/Ottawa 2500 >0.5s	0.00525	1.7463	0.00134	0.43206	0.0235	5.0647	-0.0005	0.0843
Toronto 500 <0.5s	0.0003	0.06039	-0.00017	-0.05255	-0.0005	-0.06210		-0.0073
Toronto 500 >0.5s	0.00058	0.11278	0.00011	0.03476	0.0007	0.08550		-0.008
Toronto 2500 <0.5s	0.00068	0.15322	0.00047	0.1483	0.0016	0.192	-0.0002	-0.0333
Toronto 2500 >0.5s	0.00122	0.35057	0.00047	0.15168	0.0028	0.5356	0.0003	0.0413
Vancouver 500 <0.5s	0.0017	0.61234	-0.00091	-0.29241	-0.004	-0.8631	-0.0004	-0.0709
Vancouver 500 >0.5s	0.00148	0.49318	0.00054	0.17325	0.0084	1.1379	-0.0003	-0.0435
Vancouver 2500 <0.5s	0.00991	3.11705	0.00153	0.48549	0.0184	-2.3297	-0.0006	0.1108
Vancouver 2500 >0.5s	0.00438	1.13435	-0.00089	-0.28211	0.0062	1.2744	0.0004	0.077

Table 6.10 2% in 50 year Ground Motions matching Soil Types A-E					
Montreal/Ottawa	Magnitude	Distance	Scale Factor	Duration (s)	Ag max cm/s ²
Short period soil type A	6	50	1.1	12.40	198.15
Long period soil type A	7	50	0.3	20.55	231.14
Short period soil type B	6	50	1.2	12.40	198.15
Long period soil type B	7	70	0.5	24.07	295.3
Short period soil type C	6	50	1.5	12.40	198.15
Long period soil type C	7	50	0.6	20.55	231.14
Short period soil type D	6	50	1.7	12.40	198.15
Long period soil type D	7	50	0.8	20.55	619.76
Short period soil type E	6	50	2	12.40	235.69
Long period soil type E	7	50	1.2	20.55	231.14

Table 6.11 2% in 50 year Ground Motions matching Soil Types A-E					
Toronto	Magnitude	Distance	Scale Factor	Duration (s)	Ag max cm/s ²
Short period soil type A	6	50	0.4	12.40	198.15
Long period soil type A	7	40	0.1	18.79	824.84
Short period soil type B	6	50	0.5	12.40	198.15
Long period soil type B	7	40	0.1	15.27	791.25
Short period soil type C	6	50	0.6	12.40	198.15
Long period soil type C	7	40	0.2	18.79	824.84
Short period soil type D	6	50	0.8	12.40	198.15
Long period soil type D	7	20	0.1	18.79	1589.69
Short period soil type E	6	50	1.3	12.40	198.15
Long period soil type E	7	40	0.4	18.79	824.84

Table 6.12 2% in 50 year Ground Motions matching Soil Types A-E					
Vancouver	Magnitude	Distance	Scale Factor	Duration (s)	Ag max cm/s ²
Short period soil type A	6.5	25	0.7	8.53	586.83
Long period soil type A	7.2	40	0.3	16.64	496.02
Short period soil type B	6.5	25	0.9	8.53	586.83
Long period soil type B	7.2	40	0.4	16.64	496.02
Short period soil type C	6.5	30	1.0	8.53	526.83
Long period soil type C	7.2	40	0.5	16.64	496.02
Short period soil type D	6.5	30	1.1	8.53	526.83
Long period soil type D	7.2	40	0.6	16.64	496.02
Short period soil type E	6.5	30	1.1	8.53	526.83
Long period soil type E	7.2	40	0.9	16.64	496.02

Table 6.13 Barnsdale Road Underpass Displacement Soil Types A-E						
	Displacement					
	Montreal/Ottawa		Toronto		Vancouver	
	Δ	μ	Δ	μ	Δ	μ
Short period soil type A	-0.007	-0.7916	-0.0035	0.4456	-0.0145	-1.5797
Long period soil type A	-0.0084	-0.9176	-0.0047	0.5401	-0.0157	-1.6924
Short period soil type B	-0.008	-0.907	0.004	0.5044	0.0517	6.5392
Long period soil type B	-0.0258	-2.1177	-0.0042	0.5377	0.0678	8.581
Short period soil type C	0.0138	1.7506	0.0043	0.5462	-0.0546	-6.9063
Long period soil type C	-0.0342	-4.3354	0.0072	0.9104	-0.0262	-3.3203
Short period soil type D	0.0265	3.3519	0.0049	0.6235	0.0702	8.8899
Long period soil type D	0.1562	19.7722	-0.0075	-0.8437	-0.0332	-3.5924
Short period soil type E	0.0321	4.0608	-0.0192	-0.939	0.0702	8.8899
Long period soil type E	0.2029	25.6835	0.1387	17.557	0.0625	7.9139

Table 6.14 Barnsdale Road Underpass Rotation Soil Types A-E						
	Rotation					
	Montreal/Ottawa		Toronto		Vancouver	
	θ	μ	θ	μ	θ	μ
Short period soil type A	0.00118	0.37882	0.00064	0.13312	0.00234	0.75572
Long period soil type A	0.0014	0.43936	0.00082	0.20838	0.00243	0.84836
Short period soil type B	0.00133	0.44167	-0.0007	-0.15137	-0.00818	-2.73073
Long period soil type B	0.00293	0.97098	-0.00075	-0.18278	-0.00692	-2.5247
Short period soil type C	-0.0012	-0.38555	-0.00068	-0.15322	-0.00991	-3.11705
Long period soil type C	0.00525	1.7463	0.0012	-0.36956	0.00438	1.13435
Short period soil type D	-0.00366	-1.27042	-0.00084	-0.21019	-0.01235	-3.53873
Long period soil type D	-0.02728	-10.2633	0.00124	0.41361	0.00522	2.00461
Short period soil type E	-0.00652	-1.7513	0.00143	0.44013	-0.01235	-3.53873
Long period soil type E	-0.03223	-10.1134	-0.01863	-5.55064	-0.01031	-3.06362

Table 6.15 Barnsdale Road Underpass Seat Width and Rotation Capacity						
	Seat Width (m)	$\Delta X \times 1.5$	Plastic rotational capacity of hinges θ_p	Total rotational capacity of hinges $\theta_y + \theta_p$	$0.035 + \theta_y$	Analysis value
Montreal/Ottawa 500 <0.5s	0.426	-0.00585	0.031	0.034	0.038	0.00071
Montreal/Ottawa 500 >0.5s	0.426	0.0063	0.033	0.036	0.038	0.00064
Montreal/Ottawa 2500 <0.5s	0.471	0.0207	0.039	0.042	0.038	-0.0012
Montreal/Ottawa 2500 >0.5s	0.471	-0.0513	0.029	0.031	0.038	0.00525
Toronto 500 <0.5s	0.411	0.0207	0.033	0.035	0.038	0.0003
Toronto 500 >0.5s	0.411	-0.00255	0.034	0.036	0.038	0.00058
Toronto 2500 <0.5s	0.428	0.00495	0.034	0.037	0.038	0.00068
Toronto 2500 >0.5s	0.428	0.00645	0.029	0.031	0.038	0.00122
Vancouver 500 <0.5s	0.491	0.0108	0.022	0.024	0.038	0.0017
Vancouver 500 >0.5s	0.491	0.0162	0.023	0.026	0.038	0.00148
Vancouver 2500 <0.5s	0.566	0.0159	0.017	0.02	0.038	0.00991
Vancouver 2500 >0.5s	0.566	-0.0819	0.021	0.023	0.038	0.00438

Table 6.16 CNR Overhead EBL Seat Width and Rotation Capacity						
	Seat Width (m)	$\Delta X \times 1.5$	Plastic rotational capacity of hinges θ_p	Total rotational capacity of hinges $\theta_y + \theta_p$	$0.035 + \theta_y$	Analysis value
Montreal/Ottawa 500 <0.5s	0.436	0.01125	0.053	0.058	0.040	-0.0013
Montreal/Ottawa 500 >0.5s	0.436	0.0159	0.057	0.062	0.040	-0.0018
Montreal/Ottawa 2500 <0.5s	0.481	-0.033	0.067	0.072	0.040	0.0032
Montreal/Ottawa 2500 >0.5s	0.481	-0.0825	0.049	0.054	0.040	0.0235
Toronto 500 <0.5s	0.42	-0.03	0.056	0.061	0.040	-0.0005
Toronto 500 >0.5s	0.42	0.0045	0.058	0.063	0.040	0.0007
Toronto 2500 <0.5s	0.437	-0.006	0.058	0.063	0.040	0.0016
Toronto 2500 >0.5s	0.437	-0.0135	0.049	0.054	0.040	0.0028
Vancouver 500 <0.5s	0.501	0.0198	0.037	0.042	0.040	-0.004
Vancouver 500 >0.5s	0.501	0.0441	0.04	0.045	0.040	0.0084
Vancouver 2500 <0.5s	0.578	-0.051	0.03	0.035	0.040	0.0184
Vancouver 2500 >0.5s	0.578	-0.1575	0.036	0.041	0.040	0.0062

Figure 6.1 Barnsdale Road Underpass Node Placement

Figure 6.2 CNR Overhead EBL Node Placement

Figure 6.3 CNR interior column bent

Figure 6.4 NONSPEC Single Degree of Freedom Model

Figure 6.5 Montreal/Ottawa 10% In 50 Year Short Period Displacement Time History

Figure 6.6 Montreal/Ottawa 10% In 50 Year Long Period Displacement Time History

Figure 6.7 Montreal/Ottawa 2% In 50 Year Short Period Displacement Time History

Figure 6.8 Montreal/Ottawa 2% In 50 Year Long Period Displacement Time History

Figure 6.9 Toronto 10% In 50 Year Short Period Displacement Time History

Figure 6.10 Toronto 10% In 50 Year Long Period Displacement Time History

Figure 6.11 Toronto 2% In 50 Year Short Period Displacement Time History

Figure 6.12 Toronto 2% In 50 Year Long Period Displacement Time History

Figure 6.13 Vancouver 10% In 50 Year Short Period Displacement Time History

Figure 6.14 Vancouver 10% In 50 Year Long Period Displacement Time History

Figure 6.15 Vancouver 2% In 50 Year Short Period Displacement Time History

Figure 6.16 Vancouver 2% In 50 Year Long Period Displacement Time History

Figure 6.17 CNR Montreal/Ottawa 10% In 50 Year Short Period Displacement Time History

**Figure 6.18 CNR Montreal/Ottawa 10% In 50 Year Long Period Displacement
Time History**

Figure 6.19 CNR Montreal/Ottawa 2% In 50 Year Short Period Displacement Time History

Figure 6.20 CNR Montreal/Ottawa 2% In 50 Year Long Period Displacement Time History

Figure 6.21 CNR Toronto 10% In 50 Year Short Period Displacement Time History

Figure 6.22 CNR Toronto 10% In 50 Year Long Period Displacement Time History

Figure 6.23 CNR Toronto 2% In 50 Year Short Period Displacement Time History

Figure 6.24 CNR Toronto 2% In 50 Year Long Period Displacement Time History

Figure 6.25 CNR Vancouver 10% In 50 Year Short Period Displacement Time History

Figure 6.26 CNR Vancouver 10% In 50 Year Long Period Displacement Time History

Figure 6.27 CNR Vancouver 2% In 50 Year Short Period Displacement Time History

Figure 6.28 CNR Vancouver 2% In 50 Year Long Period Displacement Time History

Figure 6.29 Montreal/Ottawa 2% In 50 Year UHS Soil Type A-E

Figure 6.30 Toronto 2% In 50 Year UHS Soil Type A-E

Figure 6.31 Vancouver 2% In 50 Year UHS Soil Type A-E

Chapter 7

7. Summary and Conclusion

7.1 Summary and Conclusion

The seismic design provisions of NBCC 1995 and 2005, AASHTO and CHBDC 2000 have been compared. The magnitude of the 2% in 50 year UHS are found to be larger than the scaled spectrum approach used in CHBDC 2000 and NBCC 1995 for periods less than one second in Montreal/Ottawa, Toronto and Vancouver. After the one second period, the 2% in 50 year UHS spectral values are found to be less than the previously used NBCC and CHBDC values. The NONSPEC program has been used to create constant η (strength-inertia ratio) displacement ductility spectra and constant strength ductility spectra for Montreal/Ottawa, Toronto and Vancouver. The displacement Ductility spectra have been created using ground motions compatible with the 10% in 50 year and 2% in 50 year UHS for Montreal/Ottawa, Toronto and Vancouver. The effects of recent changes in the seismic design methods of NBCC 2005 and ATC/MCEER 2003 on two Canadian Highway bridges are studied. The analysis method in this study has been adapted from NBCC 2005 and ATC/MCEER 2003. The 2% in 50 year UHS is used to define the seismic hazard, using the NBCC 2005 method to construct the hazard spectrum. The seismic design and analysis requirements are determined using performance objectives defined in ATC/MCEER 2003. The life safety performance objective has been chosen for the bridges in this study. Seismic design and analysis procedure E, as proposed in ATC/MCEER 2003 has been used to analyze the bridges. The effect of adopting the soil types and factors used in NBCC 2005 has been studied. The reference ground condition (soil type C) is used for the first stage of the

analysis, while the effects of using UHS matching soil type A, B, D and E are studied in the second stage of the analysis. The displacement and rotational ductility demands determined from the analysis are compared to force modification factors (R factors) recommended by ATC/MCEER 2003. The plastic hinge length, required seat width and rotational capacity of the bridge columns are evaluated based on the seismic design guidelines proposed by ATC/MCEER 2003. The effect of using 2% in 50 year UHS on magnitude of the displacement and rotation response has been studied. The effect of using 2% in 50 year UHS compatible with Soil types A to E as adopted in NBCC 2005, on the magnitude of the displacement and rotation response are studied. The displacement and rotation response are evaluated using guidelines recommended in ATC/MCEER 2003. A summary of the conclusions found in the analyses are as follows:

- The magnitude of the 2% in 50 year UHS is greater than the previously used NBCC 1995 and CHBDC 2000 spectral values for period values less than one second in Montreal/Ottawa, Toronto and Vancouver.
- For periods greater than one second the 2% in 50 year UHS values have been found to be less than the previously used NBCC and CHBDC code values.
- The short period displacement ductility demand (<0.5 seconds) for the 2% in 50 year UHS are significantly greater than those for the 10% in 50 year UHS.
- Displacement ductility spectra calculated using the NONSPEC program indicated that the response to the Montreal/Ottawa and Toronto 10% in 50 year ground motions are elastic. Montreal/Ottawa and Toronto 2% in 50 year ground motions create inelastic response in the short period range but elastic response after a

period of one seconds. Response is inelastic to the Vancouver 10% in 50 year and 2% in 50 year ground.

- The longitudinal displacement response found using the NONSPEC program is found to be within 30 to 40 percent of the results of the NEABS program.
- The seismic response in the transverse direction is less than that in the longitudinal direction for both structures and all load cases.
- The longitudinal response of the Barnsdale Road Underpass and CNR Overpass are observed to be non-linear for the Montreal/Ottawa 2% in 50 year short and long period ground motions and the Vancouver 10% in 50 year and 2% in 50 year short and long period ground motions.
- The maximum base response modification factor are calculated for each ground motion and are found to equal 6.0 except for Vancouver soil type E where $R=4.29$.
- The calculated displacement and rotational ductility values are under 6 for all cases except the Vancouver 2% in 50 year short period compatible ground motion, which is found to be 6.9. The largest response are to the Montreal/Ottawa 2% in 50 year long period ground motion and the Vancouver 2% in 50 year short and long period ground motion.
- The minimum required seat width recommended by ATC/MCEER are provided for both the Barnsdale Road Underpass and CNR Overpass
- The maximum column rotation are found to be less than recommended value for all cases.

- The displacement ductility demand for the Montreal/Ottawa long period ground motions for soil type D and E are found to be greater than the recommended value.
- The displacement ductility demand for the Toronto long period ground motions for soil type E have been found to be greater than the recommended value.
- The displacement ductility demand for Vancouver short period soil type B, C, D and E and the long period soil type B and E are found to be greater than the recommended value.
- The rotational ductility demand for the Ottawa long period ground motions for soil type D and E have been found to be greater than the recommended value.

7.2 Recommendations for Future Study

Performance objectives for Canadian Highway bridges which depend on the desired damage and service level of the bridge structure after seismic loading should be developed. The comparative increase between the ductility values found in this study and the ductility values found using CHBDC 2000 compatible ground motions should be investigated. The number of bridge structures studied should be expanded to include typical structures throughout Canada. The current study includes structures only from Ontario. The effect of boundary conditions on the response of the bridges to UHS compatible ground motions should be studied. The effect of the column height on the seismic response of the bridge should be studied to determine the optimal relation between column height, cost and displacement/rotation response.

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