# Comparisons of International Seismic Code Provisions for Bridges

By Aguiade Drak El Sebai

February 2009

Department of Civil Engineering and Applied Mechanics

McGill University

Montreal, Quebec

Canada

A thesis submitted to the graduate and Postdoctoral Studies Office in Partial fulfilment of the requirements of the degree of Master of Engineering

# ABSTRACT

This thesis presents a comparison between the AASHTO-2004, BSI-EN1998-2:2005, NBCC-2005 and the 2007 proposed AASHTO LRFD seismic design provisions with that of the 2006 CSA S6 Canadian Highway Bridge Design Code (CHBDC). A regular 2 span-90m long bridge was used to apply the seismic design loads determined from the codes investigated. The superstructure consists of a 2-lane prestressed concrete box girder, supported by a single column and two abutments. The single column is a 2400 mm circular pier with two different amounts of longitudinal reinforcements. One column had 36-45M longitudinal bars, transversely tied with 15M bars spaced at 50 mm. The second column contained 36-55M bars. The design of the bridge was carried out for three different seismic regions, Montreal, Toronto and Vancouver. In addition, the soil conditions were assumed to match those of NEHRP Soil Profile Types B and E. The research compared the effects of the seismic design spectra and overstrength factors in developing the design moments, shears and displacement ductility demands of the bridge.

This research provides recommendations for updating the current CHBDC seismic design provisions.

# RÉSUMÉ

Cette thèse présente une comparaison entre les normes « AASHTO-2004 », « BSI-EN1998-2:2005 », « CNB-2005 » ainsi qu'une comparaison entre les exigences de la nouvelle norme « AASHTO LRFD 2007 » et la norme « CSA-S6 2006 » du Code Canadien sur le Calcul des Ponts.

Un pont régulier de deux travées ayant une longueur de 90 m à fait l'objet d'une analyse conformément aux exigences des divers normes et codes relatives à la conception parasismique. La superstructure consiste en un système composé d'une poutre-caisson en béton précontraint alors que l' infrastructure consiste d'un poteau et de deux culées. Le poteau (de diamètre de 2400 mm) avait deux quantités différentes d'armature longitudinales (36 barres - 45M et 36 barres - 55M) avec des armatures transversales ayant un espacement de 50 mm. Le pont à été conçu selon les exigences parasismiques de trois divers zones sismiques dont : Montréal, Toronto et Vancouver. Les conditions de sol de types B et E de la norme « NEHRP » ont été utilisées lors de l'analyse. De plus, cette étude a comparé l'effet de divers spectres sismiques et facteurs de surcharge lors du calcul des moments et des efforts de cisaillement ainsi que lors de l'analyse de la ductilité de la structure.

Enfin, ce projet de recherche propose des recommandations pour mettre à jour les provisions parasismiques du Code Canadien sur le Calcul des Ponts.

# ACKNOWLEDGMENTS

The author would like to express his profound gratitude and appreciation to Professor Denis Mitchell for his scholarly advice, guidance and supervision. He would also like to extend his gratitude to Dr. William Cook for his timely assistance and support during the writing of this thesis.

The financial support of the National Sciences and Engineering Research Council of Canada (NSERC) is greatly appreciated.

The author is forever thankful for the assistance of his fellow research engineers, Dr. Hassan Aoude and for the unconditional love and support of his family.

# TABLE OF CONTENTS

# LIST OF TABLES

# LIST OF FIGURES

# Chapter 1 Introduction and Literature Review

## 1.1 Introduction

The purpose of this thesis is to compare the current seismic design provisions of the CSA S6-06 Canadian Highway Bridge Design Code-CHBDC (CSA, 2006) with that of the American Association of State Highway and Transportation Officials-AASHTO LRFD Bridge Design Specifications-S.I. Units (AASHTO, 2004), the proposed AASHTO provisions prepared by Roy A. Imbsen for the Subcommittee for Seismic Effects on Bridges T-3 (Imbsen, 2007), National Building Code of Canada (NRCC, 2005) and the British Standard-Eurocode 8 - Design of Structures for Earthquake Resistance - Bridges (BSI, 2005), and to examine the current approach used in the CHBDC for seismic design of bridges. The main components of the comparisons are based on the seismic design spectra and the effects of the ductility and overstrength factors on the seismic design loads. For a quantitative verification of the effects of these parameters, a 2 span prestressed concrete box girder bridge, supported by a single reinforced concrete column and two abutments will be used. To enable the evaluation of the bridge under linear dynamic analysis, the bridge will be modelled in SAP2000. Representative seismic ground motions for Montreal, Toronto and Vancouver are used to analyze the same bridge

(Adams and Atkinson, 2003). Vibration modes will be summed up so that the effective modal mass amount to 90% of the total mass of the bridge.

Seismic design provisions are updated according to experimental research, observations of structural behaviour following major seismic events and the design provisions of national and international codes (CSA, 2006; NRCC, 2006; Heidebrecht, 2003). Close collaboration in research between Canada and the U.S.A justifies the close affiliation in the development of the seismic design provisions in the CHBDC with that of the 1994 AASHTO LRFD Seismic Provisions (CSA, 2006). Chapter 1 reviews the main seismic design provisions of the CHBDC and AASHTO LRFD code.

## 1.2 Summary of the CHBDC Seismic Design Provisions

### 1.2.1 Seismic Design Philosophy

The CHBDC adopted the same seismic design philosophy of capacity design as in the AASHTO LRFD Specifications to achieve the desired performance criteria (CSA, 2006; AASHTO, 2004). In the seismic design of structures, capacity design is utilized to dissipate the energy resulting from the ground motion through a hierarchical design approach together with careful detailing of the ductile sub-structural elements. This method avoids the need to design for the large elastic seismic forces by allowing local structural damage resulting from inelastic hinging. The capacity design of concrete bridges dictates that the substructures perform as the main energy dissipating structural components while keeping the remaining components (capacity protected elements) elastic. Special detailing and design provisions ensure that the strength of the structure is maintained during and after the earthquake event. Proper rehabilitation procedures are undertaken for the damaged structural components following a major event. Force-based capacity design involves the design of the energy dissipating elements for the elastic seismic load resulting from a seismic event adhering to a particular probability of exceedance, reduced by the response modification factor. The response modification factor accounts for the ductility and redundancy of the structure and is based on past observations of structural behaviour during earthquakes, the development of international codes and experimental research. As brittle failures offer little advanced warning, shear failures in all structural components must be avoided. An amplification factor on the nominal and probable moment capacities of the energy dissipating component is used to ensure appropriate safety against all brittle modes of failure (e.g., shear failures). The remaining structural components are designed to remain elastic. This design approach results in a structure that is stronger than the seismic loads it was initially designed for (Mitchell and Paultre, 1994).

Seismic design and detailing provisions ensure adequate concrete confinement and prevent brittle failures, joint failures and buckling of reinforcing bars. Confining the concrete improves its compressive strength and more importantly the ductility as shown by the stress-strain relationships of confined concrete specimens (Mander et. al, 1988). Structures can therefore be designed with different values of response modification factors, and exhibit different structural behaviour under seismic loads. Member design and detailing therefore play a major role in influencing the structure's overall ductility.

### 1.2.2 Seismic Design Parameters

The main seismic design parameter that reflects the seismic hazard at a site is the elastic seismic response coefficient, $C_{sm}$. The elastic seismic response coefficient (CSA, 2006) at each vibration mode is determined as follows:

$$C_{sm} = \frac{1.2AIS}{T_m^{\;2/3}} \leq 2.5AI \quad (\leq 2.0AI \;\; For\ Soil\ \mathrm{Pr}ofile\ Types\ III\ or\ IV\ with\ A \geq 0.3) \qquad [1.1]$$

The elastic seismic response coefficient for Soil Profile Types III or Type IV soils, for modes other than the fundamental mode and with periods less than 0.3s is taken as:

$$C_{sm} = AI(0.8 + 4.0T_m) \qquad [1.2]$$

Where

A = firm ground zonal acceleration ratio specified for an event with a 475 year return period (i.e., 10% in 50 years probability of exceedance)

S = site coefficient, values of which are given in Table 1.1 for different Soil Profile Types. The S factor incorporates the effects of the site conditions on a bridge in an earthquake, by adjusting the firm ground horizontal accelerations.

Table 1.1: CHBDC - Site Coefficients

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

The above site coefficient values were adopted from the AASHTO LRFD code. The Soil Profile Types considered in the CHBDC and AASHTO LRFD codes are determined qualitatively. They include four Soil Profiles ranging from rock or stiff soils to soft soil

$T_m$ = period of vibration of the $m^{th}$ mode (seconds)

I = Importance factor based on the importance category

= 3.0 for lifeline bridges. The design load surpasses the elastic limit when the value of the Importance factor exceeds that of the response modification factor. Consequently, the Importance factor should be limited by the value of the response modification factor (Mitchell et al., 1998)

= 1.5 for emergency-route bridges

= 1.0 for other bridges

The elastic seismic coefficient for long (T > 4.0 s) periods must not be less than:

$$C_{sm} = \frac{3AIS}{T_m^{\frac{4}{3}}} \quad [1.3]$$

The elastic seismic coefficient for short periods has a plateau at:

$$C_{sm} = 2.5AI \quad [1.4]$$

The elastic seismic base shear is equal to the product of the elastic seismic coefficient and the effective weight of the bridge. The effective weight of the bridge can be based on the superstructure's weight and a third of the piers' weights. Except for the inclusion of the importance factor, the elastic seismic response coefficient equation is identical to that of AASHTO 1994. In the linear dynamic analysis method, the elastic seismic loads are found for each vibration mode, and the resulting forces are added using the Complete Quadratic Combination method (CQC) or the Square Root of the Sum of the Squares (SRSS) method, until 90% of the mass participating ratio is attained. This approach is not entirely accurate, because squaring the elastic forces at each mode eliminates the effect of the load's direction. The seismic load in each of the horizontal principal axes is determined, and then 100% and 30% of the absolute values in each direction are combined. The maximum result of the aforementioned combinations is used to design the bridge for seismic loads. The effects of the vertical ground motion are accounted for by using 0.8 and 1.25 dead load factors in combination with the seismic loads.

In 1994, AASHTO developed their seismic design spectra from the National Earthquake Hazards Reduction Program (NEHRP) document "Recommended Provisions for the Development of Seismic Regulations for Buildings" (CSA, 2006). The NEHRP document used the Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV) to define the design spectra. AASHTO modified the recommended provisions by depending on the PGA only in developing its seismic design spectra. In 2000, the CHBDC seismic sub-committee adopted the same approach undertaken by AASHTO, and based the zonal acceleration on the 1995 NBCC Peak Horizontal Ground Acceleration (CSA, 2000; 2006). The PHA values are based on statistical analyses of historical earthquakes and assumed attenuation equations, and represent an earthquake event with a 10% probability of exceedance in 50 years (i.e., a return period of 475 years). It is noted that the seismic response coefficient in the CHBDC is based on the mean accelerations of many generated ground motions, and therefore represent mean spectral shapes (Naumoski et al., 2000). The CHBDC seismic subcommittee was aware of new design approaches (e.g., Uniform Hazard Spectra) in defining ground motion incorporated in the 2005 NBCC (NRCC, 2005), but the subcommittee decided to keep the approach used in AASHTO (CSA, 2006; Mitchell et. al., 1998). It is interesting to note that while Montreal and Vancouver have very different ground motion characteristics; both have the same PHA value of 0.2g, which results in the same seismic design loads when all other factors are kept constant (CSA, 2006; Heidebrecht, 2003; Adams and Atkinson, 2003). It is also interesting to consider the relevance of the 1.2 factor adopted by both codes, perhaps implemented to overcome uncertainties in the nature of the design load and the method used to calculate it.

Research by Naumoski et al. (2000) showed that the CHBDC seismic response coefficient compares favourably with ground motions having a high to intermediate acceleration/velocity (a/v) ratios (a characteristic of eastern Canada), but poorly against ground motions with low a/v ratios (a characteristic of western Canada), and strong earthquakes with large epicentral distances. This is because the dependence on the PHA as the main seismic design parameter represents earthquake motions with $a \cong v$. The solution proposed by Naumoski et al. (2000) to this inconsistency was to increase the seismic response coefficient by 50% (i.e., $C'_{sm}$=1.5 $C_{sm}$) for locations with low a/v ratios. Naumoski et al. (2000) found that the CHBDC seismic response spectra envelopes the Geological Survey of Canada (GSC) Uniform Hazard Spectra for the eastern and western regions of Canada. They also found that the CHBDC seismic coefficient was consistent with that of the 1994 Eurocode, 1999 CALTRANS and 1994 Transit New Zealand seismic provisions. The anticipated changes to the seismic design spectra have been realized in various design codes, and this thesis compares the 2006 CHBDC response spectra with those of other codes.

### 1.2.3 Response Modification and Importance Factors

The seismic provisions in AASHTO and the CHBDC are based on the design philosophy mentioned above, whereby the elastic seismic loads are reduced by a response modification factor. The response modification factor-R reflects the sub-structure's ability to dissipate energy and its redundancy (CSA, 2006). The response modification factor, also known as the ductility factor, is defined as follows:

$$R = \mu_d = \frac{u_{\max}}{u_y} \qquad [1.5]$$

Where:

$R, \mu_d$ = response modification factor or ductility factor

$u_{max}$ = maximum displacement of the nonlinear system

$u_y$ = displacement of the system at yielding

Dynamic analyses verified the proximity of the maximum displacements of linear and non linear structures, especially for relatively long natural periods (Blume et al., 1961). Structures with short periods however, experience higher inelastic demands than those estimated by the Equal Displacement method (MCEER/ATC, 2003). The Equal Energy method is therefore more accurate for short period (e.g., T<0.7s) structures than the equal displacement method

(CSA, 2006). The above findings partly explain the conservatism in the CHBDC R values, and the development of new equations for the ductility factor incorporating the effects of the natural period of the structure by various codes (CSA, 2006; NCHRP 2001; BSI, 2005). Those developments are particularly critical for reinforced concrete sections, given the decrease in its energy dissipating capabilities associated with stiffness reduction under earthquake induced load reversals (Park and Paulay, 1975). The current CHBDC provisions do not account for the effects of the natural period of the structure on the response modification factor, or the determination of the inelastic displacement (CSA, 2006).

In line with the capacity design method, capacity protected members (e.g., joints, cap beams and superstructure) are designed for the lesser of the forces resulting from the substructure's inelastic deformations, or the elastic seismic loads developed with R = 1.0 and I = 1.0. In line with AASHTO's seismic provisions, CHBDC dictates the determination of the probable flexural resistance (inelastic hinging moments) of concrete and steel sections by multiplying their nominal flexural resistances ($M_n$) by 1.30 and 1.25, respectively. These factors are attributed to the difference between the probable material strengths and the specified ones. Differences arise from the variability in the material microstructure, strain hardening and enhanced behaviour response resulting from confinement. Connectors are designed for the lesser of 1.25 times the elastic seismic forces, and the forces developed by the ductile substructure attaining 1.25 times its probable resistance (CSA, 2006). Table 1.2 shows the response modification factors used in the 2006 CSA CHBDC:

Table 1.2: CHBDC - Response Modification Factors

| **Ductile Substructure Elements** | **Response Modification Factor, R** |
|---|---|
| Wall-type piers in direction of larger dimension | 2.0 |
| Reinforced concrete pile bents | |
| Vertical piles only | 3.0 |
| With batter piles | 2.0 |
| Single columns | |
| Ductile reinforced concrete | 3.0 |
| Ductile steel | 3.0 |
| Steel or composite steel and concrete pile bents | |
| Vertical piles only | 5.0 |
| With batter piles | 3.0 |
| Multiple-column bents | |
| Ductile reinforced concrete | 5.0 |
| Ductile steel columns or frames | 5.0 |
| Braced frames | |
| Ductile steel braces | 4.0 |
| Nominally ductile steel braces | 2.5 |

The response modification factors above are consistent with the equivalent ductility factors in 2004 AASHTO LRFD, if the importance factor is taken as 1.0 (Mitchell et. al 1998). The CHBDC provisions provide more response modification factors for emergency-route and lifeline bridges than AASHTO. Compared with AASHTO, the CHBDC provisions assume a conservative design approach for less ductile systems, to discourage the use of such systems in resisting seismic loads (CSA, 2006; Mitchell et. al., 1998).

Structures differ in their uses and importance to society. To distinguish between performance requirements of different structures, importance factors are used. As with the AASHTO specifications, bridges in the CHBDC are classified according to social/survival and security/defence requirements (CSA, 2006; AASHTO, 2004). The allowable level of damage in a bridge from an earthquake varies with the structure's importance to society. The performance requirements and permissible damage levels in the 2004 AASHTO LRFD and 2006 CHBDC provisions are shown in Table 1.3:

Table 1.3: CHBDC & AASHTO Performance Requirements

| Earthquake Size | Bridge Importance Classification | | |
|---|---|---|---|
| | **Critical/Lifeline** | **Essential/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 (e.g.,1000 or 2500-year return period)** | Emergency vehicles Immediate use | Repairable damage | No collapse |

The performance criteria are met by adhering to the capacity design and detailing provisions prescribed in each code, including the implementation of the response modification factor. As the Importance factors increase above 1.0, then the design loads approach the elastic seismic loads. It is evident from Eq. 1.1 that the multiplication of the Importance factor with the zonal acceleration is meant to ensure that the expected performance criteria are met. To attain the required performance criteria, AASHTO included the importance factor in the response modification factor as shown in Table 1.4:

Table 1.4: AASHTO 2004 Response Modification Factors

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

The response modification factor in the 2004 AASHTO LRFD provisions is equal to the quotient of the R factor found in the CHBDC, and the Importance factor, $R_{AASHTO} = \frac{R_{CHBDC}}{I}$ (Mitchell et. al, 1998). As with the AASHTO provisions, the design methods in the CHBDC are allocated according to the zonal acceleration, the regularity of the bridge and the Importance factors. The methods currently used are the Single-Mode Spectral method, Multi-Mode Spectral method, Time History method and Uniform Load method. Table 1.5 shows the analysis requirements for multi-span bridges:

Table 1.5: CHBDC Analysis Requirements for Multi-Span Bridges

| Seismic Performance Zone | Multi-span Bridges | | | | | |
|---|---|---|---|---|---|---|
| | Lifeline Bridges | | Emergency-route bridges | | Other bridges | |
| | Regular | Irregular | Regular | Irregular | Regular | Irregular |
| 1 | * | * | * | * | * | * |
| 2 | MM | MM | UL | MM | UL | SM |
| 3 | MM | TH | MM | MM | UL | MM |
| 4 | MM | TH | MM | MM | SM | MM |

* = no seismic analysis required

UL = Uniform-Load method. This method is equivalent to the Static method whereupon the fundamental vibration mode is the controlling bridge response to an earthquake. A drawback to this method is that it overestimates transverse shears at abutments

SM = Single-Mode Spectral method. This method is also consistent with the Static method

MM = Multi-Mode Spectral method. This method is based on the linear dynamic analysis of the bridge, where each vibration mode is determined from the elastic response spectrum, and then summed up by the Complete quadratic Combination (CQC) or Square Root of the Sum of the Squares (SRSS) method

TH = Time - History method. This method is used for bridges with complex geometries and those close to earthquake faults, among other factors

The seismic performance zone is categorized according to the zonal acceleration. In modelling the ductile structural elements, un-cracked sections will yield conservative seismic loads but inaccurate displacements. Cracked sectional properties will therefore be used for the ductile elements in this thesis.

## 1.3 Objectives of this Research Project

The objective of this research program is to compare the seismic design provisions of various codes with the current CHBDC. A sample bridge will be used to apply the different seismic design provisions. The following aspects will be examined:

- Review and compare the seismic design provisions in the 2006 CHBDC, 2004 AASHTO, 2005 British Standard BSI – Eurocode 8, 2005 NBCC and proposed AASHTO LRFD code.
- Determine the seismic design loads on a regular 2-span 90m long prestressed concrete bridge, supported by a single reinforced concrete column and two abutments. The design spectra and the response modification factors of the aforementioned design codes, along with the seismic design parameters from Montreal, Toronto and Vancouver, will be used to determine the seismic design loads.
- Compare the seismic design loads developed from the codes studied.
- Evaluate the impact of the spectral accelerations and the overstrength factors on the seismic design loads.
- Recommend updates for the current CHBDC seismic design provisions as warranted by the results of this research project.

# Chapter 2 National and International Seismic Design Code Provisions

## 2.1 Seismic Design Spectra

A seismic design spectrum is a graph depicting the averaged values of a design parameter (e.g., spectral acceleration-SA), as a function of the natural period of a single degree-of-freedom (SDOF) system (Newmark and Rosenblueth, 1971). The PHA parameter represents the ground acceleration, and SA represents the acceleration of a mass in a linear single degree-of-freedom system. The Uniform Hazard Spectra (UHS) is defined by “spectral acceleration ordinates at different periods calculated at the same probability of exceedance” (NRCC, 2006; Heidebrecht, 2003). Earthquakes with different ground motion characteristics (e.g., magnitudes and distances from site) are used to construct the UHS. This is different from the Response Spectrum (RS) which is based on the effect of one earthquake event on different SDOF systems. The UHS has been adopted by the Geological Survey of Canada (GSC), because it provides a consistent hazard level for structures using site specific characteristics (Heidebrecht, 2003; Adams and Atkinson, 2003). Single level bridges can be closely approximated as SDOF structures, therefore enhancing the advantages of using the SA parameter in lieu of the ground PHA.

The following sub-sections review the design spectra according to the 2005 NBCC, 2005 BS-EN 1998 and the proposed AASHTO LRFD seismic provisions.

### 2.1.1 Seismic Design Spectrum in the 2005 NBCC

Guided by the Canadian National Committee on Earthquake Engineering (CANCEE), the 2005 NBCC underwent significant changes from the preceding editions. Unlike the provisions of earlier editions, the 2005 NBCC implements the UHS in defining seismic hazards. The 2005 NBCC-UHS is constructed using the 5% damped spectral accelerations that have a probability of exceedance of 2% in 50 years (i.e., a return period of approximately 2500 years), and are determined probabilistically at a median confidence level. The Probabilistic approach is replaced by the Deterministic approach to estimate the generated ground motions from the Cascadia Subduction Zone. The GSC used the scaled-spectrum approach to develop the design spectra prior to the adoption of the UHS. The scaled-spectrum approach implemented a standard spectral shape based on the California ground motion characteristics. Given the variance of the earthquake characteristics among regions, this approach resulted in conservative design parameters when applied to regions in Canada (Adams and Atkinson, 2003). The GSC provides spectral accelerations at a 2% in 50 years probability of exceedance for various geographical locations across Canada, and therefore eliminates the shortcomings in the scaled-spectrum method (NRCC, 2006; Adams and Atkinson, 2003).

The 1997 NEHRP provisions (Building Seismic Safety Council) specified design earthquake ground motion at a probability of exceedance of 2% in 50 years, to provide a more uniform margin against collapse. Adams and Halchuk (2003) also show that this low level of probability ensures a consistent level of safety across Canada. The UHS has therefore enabled the comparison of seismic design forces across Canada, and the elimination of the seismic response factor-S, which was used in earlier editions of the NBCC to account for different a/v ratios at different locations. The UHS method can overcome the shortcomings found in the CHBDC design spectrum, which depends solely on the PHA, and therefore underestimates the effects of seismic hazards with low a/v ratios (Naumoski et al., 2000). Other factors that reinforced the adoption of the UHS approach include improved understanding of seismic hazards, seismotectonics and improved ground motion relations for eastern and western regions of Canada (NRCC, 2006). The GSC preserved the level of safety in historically high seismic regions, and ensured an adequate safety level for regions with historically low seismic activities (Adams and Atkinson, 2003).

The design ground motions are specified at the median level, unlike the approach of the U.S. Geological Survey (USGS) which uses the mean values.

The NRCC adopted the median values because the mean values include the unreliable measure of the epistemic uncertainty (e.g., uncertainty in modelling assumptions, extrapolation of data, etc.) (Adams and Atkinson, 2003). The use of the median values implies that there is a 50% chance that the design ground motion will be exceeded. This is one reason why structures designed using the 2005 NBCC spectra should have sufficient reserve capacity for the nonlinear deformations (NRCC, 2006). It is worth noting that the US building codes stipulate the adjustment of the mean hazard values by 2/3, to develop the design loads (Adams and Atkinson, 2003). The 2005 NBCC provisions define the minimum lateral earthquake force as follows:

$$V = \frac{S(T_a)M_v I_E W}{R_d R_o} \geq \frac{S(2.0)M_v I_E W}{R_d R_o} \tag{2.1}$$

$$V_{\max} \leq \frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o} \quad For\ an\ SFRS\ with\ R_d \geq 1.5 \tag{2.2}$$

Where

$S(T_a)$ = design spectral response acceleration at the fundamental period $T_a$

$S(T)$ = $F_aS_a$ (0.2) for T less than or equal to 0.2s

= min. of $F_vS_a$ (0.5) or $F_aS_a$ (0.2), for T = 0.5s

= $F_vS_a$ (1.0) for T=1.0s

=$F_vS_a$ (2.0) for T = 2.0s

= $F_vS_a$ (2.0)/2 for T ≥ 4.0s

The spectral accelerations are based on the NEHRP Site Class C, and the $F_a$ and $F_v$ factors are used to adjust the values for the site specific class. Unlike the CHBDC, the NBCC classifies soil profiles according to the NEHRP method, whereby the average shear wave velocity and/or the average standard penetration resistance are used. The site class factors incorporate the effects of the subsurface conditions on the structure's response to earthquakes, including the amplification and de-amplification of the seismic motions (Heidebrecht, 2003)

$M_v$ = factor to account for higher mode effect on base shear. The factor depends on the spectral shape and seismic force resisting system (SFRS)

$I$ = Importance factor. The seismic design load approach the elastic level as the Importance factor exceed 1.0, reducing the inelastic deformation demand on a structure accordingly (Humar and Mahgoub, 2003)

$R_d$ = ductility factor

$R_o$ = overstrength factor

$W$ = effective weight of the building. The effective weight includes the dead load, 25% of the design snow load, 60% of the storage load and 100% of the weight of tanks

The current provisions advocate the use of the linear dynamic analysis in lieu of the static method, given its more accurate portrayal of the structure's behaviour under the dynamic earthquake loads.

### 2.1.2 Seismic Design Spectrum in the 2005 Eurocode 8

The 2005 BS EN 1998-2:2005 provisions implement the ground acceleration ($a_{gR}$), determined at a probability of exceedance of 10% in 50 years (i.e., a return period of 475 years), in defining the seismic design spectrum. The design ground acceleration $a_g$ is determined by multiplying the reference seismic action $a_{gR}$ by an importance factor $\Upsilon_I$. The Importance factor is meant to reflect the social and economical impact of a bridge failure on society (e.g., human casualties, disruption of day-to-day activities, etc.). Bridges are generally categorized in Importance Class II, which is comparable to the CHBDC and AASHTO "Other" bridge classification. If the failure of a bridge impedes communication and results in a large number of deaths, then the bridge belongs to Importance Class III. Bridges belong to Importance Class I when the failure is not detrimental to communication, and the design level is unnecessarily conservative. The recommended values for class I, II, and III are 0.85, 1.0 and 1.3, respectively. Unlike the effects of the Importance factor in the US codes, The Eurocode implements the factor to modify the design hazard level (i.e., mean return period) (Fardis et al., 2005).

The 2005 BS EN 1998 horizontal elastic response spectrum is constructed as follows:

$$S_e(T) = a_g.S[1+\frac{T}{T_B}.(\eta.2.5-1)] \quad For\ 0 \le T \le T_B \qquad [2.3]$$

$$S_e(T) = a_g.S.\eta.2.5 \quad For\ T_B \le T \le T_C \qquad [2.4]$$

$$S_e(T) = a_g.S.\eta.2.5.\frac{T_C}{T} \quad For\ T_C \le T \le T_D \qquad [2.5]$$

$$S_e(T) = a_g.S.\eta.2.5.\frac{T_C T_D}{T^2} \quad For\ T_D \le T \le 4s. \qquad [2.6]$$

Where:

$S_e(T)$ is the elastic response spectrum

$T$ is the vibration period of a linear SDOF system

$a_g$ is the design ground acceleration on type A ground ($a_g = \Upsilon_1.a_{gR}$)

$T_B$ is the lower limit of the period of the constant spectral acceleration branch

$T_C$ is the upper limit of the period of the constant spectral acceleration branch

$T_D$ is the value defining the beginning of the constant displacement response range of the spectrum

$S$ is the soil factor. In line with the NEHRP specifications, the Eurocode 8 requires the use of the shear wave velocity or the average standard penetration resistance to classify soils. The soil classifications according to the NEHRP and Eurocode are correlated as shown in Table 2.1:

Table 2.1: Correlation between NEHRP 2000 and Eurocode 8 Soil Classes

| Design Code | Soil Class | | | |
|---|---|---|---|---|
| Eurocode 8 | A | B | C | D |
| NEHRP 2000 | A&B | C | D | E |

η is the damping correction factor with a reference value of η=1 for 5% viscous damping

The horizontal design spectrum is obtained as follows:

$$S_d(T) = a_g.S.[\frac{2}{3} + \frac{T}{T_B}.(\frac{2.5}{q} - \frac{2}{3})] \quad For\ 0 \le T \le T_B \tag{2.7}$$

$$S_d(T) = a_g.S.\frac{2.5}{q} \quad For\ T_B \le T \le T_C \tag{2.8}$$

$$S_d(T) = a_g.S.\frac{2.5}{q}.\frac{T_C}{T} \ge \beta.a_g \quad For\ T_C \le T \le T_D \tag{2.9}$$

$$S_d(T) = a_g.S.\frac{2.5}{q}.\frac{T_C T_D}{T^2} \ge \beta.a_g \quad For\ T_D \le T. \tag{2.10}$$

Where:

$S_d(T)$ is the design spectrum

q is the behaviour factor (equivalent to the CHBDC response modification factor)

β is the lower bound factor for the horizontal design spectrum. A value of 0.2 is recommended

The Eurocode 8 provisions define earthquakes with two types of spectra, one for moderate to large earthquakes having a surface-wave magnitude ($M_s$) greater than 5.5, and another for low magnitude earthquakes with ($M_s$) not greater than 5.5.The spectra were based on a large number of recorded ground motions, and therefore provide a more accurate representation of the earthquake characteristics in Europe (Fardis et al., 2005). The 2004 BS EN 1998-1 code provisions include a serviceability check based on a seismic design load with a 10% in 10 years probability of exceedance (i.e., an earthquake event with a 95 year return period). The serviceability check is meant to maintain the functionality of the bridge for more frequent earthquake events.

The 2004 Eurocode 8 for buildings adopted the linear dynamic analysis as the main design method, given the inherent dynamic characteristic of the seismic load. Differences exist however between the implementation of the linear dynamic analysis in the Eurocode 8 and many US codes. For example, the Eurocode 8 requires the modal combination of the elastic seismic loads at the level of the final seismic action effects (e.g. internal forces, displacement) (Fardis et al., 2005).

It is interesting to note that, in the low period range, the Eurocode design spectrum decreases for very low period values, unlike that of the NBCC which has a plateau within this range. This conservative approach by the NRCC overcomes the inaccuracy inherent in determining the periods for very stiff structures, and the resulting seismic design loads (NRCC, 2005). The Eurocode 8 provisions also include the definition of the vertical elastic response, the adjustment of the elastic spectra for different damping ratios and the adjustment of the displacement ductility factor with the period of the structure (BSI, 2005).

### 2.1.3 Design Spectrum in the 2007 Proposed AASHTO LRFD provisions

Imbsen (2007) proposed the inclusion of the displacement based capacity design approach, in lieu of the force based capacity design method implemented by AASHTO. The displacement demand on a bridge is determined from the design spectra, which corresponds to a seismic hazard level with a 7% in 75 years probability of exceedance (e.g., an earthquake event with a 1000 year return period). The various sources of conservatism in current design procedures, and the inclusion of a 2/3 factor on seismic hazards with a 2500 year return period validate the adoption of this hazard level. Table 2.2 lists some of the sources of conservatism (Imbsen & Associates, 2007):

Table 2.2: Sources of Conservatism in the U.S. Seismic Design Provisions

| Source of Conservatism | Safety Factor |
|---|---|
| Computational vs. Experimental Displacement Capacity of Components | 1.3 |
| Effective Damping | 1.2 to 1.5 |
| Dynamic Effect (i.e., strain rate effect) | 1.2 |
| Pushover Techniques Governed by First plastic Hinge to Reach Ultimate Capacity | 1.2 to 1.5 |

The trend towards implementing continuous superstructures improves the redundancy in bridges, adding another source of conservatism. The improved redundancy in the system corresponds to the increase in the number of plastic hinges required for failure before an overall collapse becomes imminent (Imbsen & Associates, 2007). Imbsen& Associates (2007) performed a probabilistic and deterministic study on 20 sites to evaluate the lower hazard level. The study was based on the Probabilistic Seismic Hazard Analysis (PSHA) and the Deterministic Seismic Hazard Analysis (DSHA) values at a period of one second. The PSHA/DSHA increased for the majority of sites considered at that hazard level.

Imbsen & Associates (2007) investigated the seismic design philosophies of various U.S. codes, summaries of which are included herewith:

- NEHRP 1997 seismic hazard practice is based on the findings of the Seismic Design Procedures Group (SDPG). The SDPG developed specifications that implement a uniform margin of failure, against an economically justified design level. The SDPG labelled this hazard level the Maximum Considered Earthquake (MCE), and defined it as an event corresponding to a 2% in 50 years probability of exceedance. This hazard level however, overestimated near-fault ground motions in California, and resulted in structures with considerable reserve capacity at those locations.

Consequently, the SDPG redefined the MCE as the lesser of the 2% in 50 years event determined probabilistically, and 1.5 times the mean ground motion obtained using a deterministic approach. The ultimate design loads are reduced by multiplying the MCE design values by 2/3.

- NYCDOT and NYSDOT have adopted the 1996 AASHTO provisions with modifications. Critical bridges are designed for two earthquake levels, and bridges in other categories are designed for only one. Bridges in all Importance categories are required to meet the “No Collapse” performance criteria. Table 2.3 summarizes the bridge categories and the relevant performance criteria for several counties in New York State:

Table 2.3: NYCDOT Bridge Performance Criteria

| **Importance Category** | **Seismic Hazard Level** | **Return Period** | **Probability of Exceedance** | **Performance Criteria** |
|---|---|---|---|---|
| Critical Bridges | Upper Level Safety | 2500 yrs | 2% in 50 yrs. | Repairable damage, limited access for emergency traffic within 48 hours, full service within months. |
| | Lower Level Functional | 500 yrs | 10% in 50 yrs. | No damage to primary structural elements, minimal damage to other components, full access to normal traffic available immediately. |
| Essential Bridges | One Level Safety | | 2/3 (2% in 50yrs) | Repairable damage, 1 or 2 lanes available within 72 hours, full service within months |
| Other Bridges | One Level Safety | | 2/3 (2% in 50yrs) | Significant damage. Traffic interruption. |

The DOT also reduces the design loads by a factor of 1.5 for bridges belonging to the “Essential” or “Other” Importance category.

- Table 2.4 shows the design earthquakes and the seismic performance objectives proposed in the NCHRP 12-49 seismic provisions:

Table 2.4: NCHRP 12-49 Provisions - Performance Criteria

| **Probability of Exceedance For Design Earthquake Ground Motion** | | **Life Safety** |
|---|---|---|
| Maximum Considered Earthquake (MCE)-3%PE in 75 yrs or 1.5 Median Deterministic | Service | Significant Disruption |
| | Damage | Significant |
| Expected Earthquake (EE) 50% PE in 75 yrs. | Service | Immediate |
| | Damage | Minimal |

Similar to the NEHRP 1997 provisions, the Maximum Considered Earthquake corresponds to a 3% in 75 yrs. probability of exceedance (i.e., earthquake event with a 2500 year return period), or 1.5 times the median values developed from a deterministic approach. The Expected Earthquake is consistent with the serviceability check implemented in the Eurocode 8 provisions. The designer can either check the bridge for the elastic loads from the Expected Earthquake hazard level, or depend solely on satisfying the performance requirements for the MCE.

- South California Department of Transportation (SCDOT) officials employed the USGS maps, which provide values to construct the UHS at a 2% in 50 years probability of exceedance. The SCDOT provisions differ from those of the NYCDOT and NYSDOT in that the 2/3 factor on the design earthquake is eliminated. The SCDOT officials aim at following the approach of Caltrans, and incorporate the displacement based design for reinforced concrete ductile sub-structures. Table 2.5 summarizes the seismic performance criteria adopted by the SCDOT:

Table 2.5: SCDOT Bridge Performance Criteria

| Ground Motion Level | Performance Level | Normal bridges | Essential Bridges | Critical Bridges |
|---|---|---|---|---|
| Functional-Evaluation | Service | N/A | N/A | Immediate |
| | Damage | N/A | N/A | Minimal |
| Safety-Evaluation | Service | Impaired | Recoverable | Maintained |
| | Damage | Significant | Repairable | Repairable |

The officials adopted the two hazard level approach for bridges belonging to the “Critical” Importance category only. The Functional Evaluation and the Safety Evaluation Earthquake are specified at a probability of exceedance of 10%in 50 years and 2% in 50 years, respectively.

The above summaries show that several US codes reduced the seismic loads by 1.5 when the hazard level was specified at a probability of exceedance of 2% in 50 yrs. This factor has been attributed to the reserve capacity observed in structures and sources of conservatism inherent in the design. In lieu of using this reduction factor, a lower hazard level has been proposed for the new AASHTO LRFD provisions. Imbsen (2007) divided the performance criteria into four Seismic Design Categories, namely SDC A, B, C and D. The performance criteria for SDC B, C and D are based on ensuring that the seismic displacement demand is less than the displacement capacity, and that capacity design is implemented to design the capacity protected members. The design spectrum in the proposed AASHTO LRFD provisions is developed as follows:

For periods less than or equal to $T_o$, $S_a$ is defined as follows:

$$S_a = (S_{DS} - A_s)\frac{T}{T_o} + A_s \qquad [2.11]$$

$$T_o = 0.2T_s \qquad [2.12]$$

$$T_s = \frac{S_{D1}}{S_{DS}} \qquad [2.13]$$

For periods greater than or equal to $T_o$ and less than or equal to $T_s$, $S_a$ shall be defined as follows:

$$S_a = S_{DS} \qquad [2.14]$$

For periods greater than $T_s$, $S_a$ shall be defined as follows:

$$S_a = \frac{S_{D1}}{T} \qquad [2.15]$$

Where:

$F_{pga}$ = site coefficient for peak ground acceleration

PGA = peak horizontal ground acceleration coefficient on Class B rock

$F_a$ = site coefficient for 0.2 second period spectral acceleration

$S_s$ = 0.2 second period spectral acceleration coefficient on Class B rock

$F_v$ = site coefficient for 1.0 second period spectral acceleration

$S_1$ = 1.0 second period spectral acceleration coefficient on Class B rock

$A_s$ = effective peak ground acceleration coefficient. $A_s = F_{pga}.PGA$

$S_{D1}$ = design spectral acceleration coefficient at 1.0 second period. $S_{D1}=F_vS_1$

$S_{DS}$ = design spectral acceleration coefficient at 0.2 second period. $S_{DS}=F_aS_s$

T = period of vibration (sec.)

## 2.2 Overstrength and Ductility Factors

Research by Mitchell et al. (1994, 2003) has shown that buildings designed according to capacity design result in structures much stronger than the base seismic shear it was originally designed for. The research has also shown that ductile structures can have a larger capacity than those designed for moderate or low ductility. The capacity design and detailing provisions enhance the strength and ductility of the concrete ductile members by confining concrete through strict reinforcement spacing requirements. The hierarchy of strength in a structure is established by designing capacity protected members for the probable flexural resistance of the ductile elements. The probable flexural resistance of a member accounts for the variable material strengths, the conservative detailing layouts and the construction quality. The brittle shear failure is also avoided by developing the shear load based on the probable flexural capacity of the ductile member. In the CHBDC, the probable flexural resistance is taken as $1.3M_n$ for concrete structures and $1.25M_n$ for steel (CSA, 2006). The implementation of the capacity design method and the consequent reserve capacity in the structure validated the inclusion of the overstrength factor in the 2005 NBCC design. The following sub-sections include reviews of the response modification factors in the 2005 NBCC, 2005 BS EN 1998 and the 2007 proposed AASHTO LRFD seismic provisions.

### 2.2.1 Overstrength and Ductility Factors in the 2005 NBCC

The overstrength factor, $R_o$, for reinforced concrete in the 2005 NBCC (Mitchell et al., 2003) is defined as follows:

$$R_o = R_{size}\ R_{\varphi}\ R_{yield}\ R_{sh}\ R_{mech} \quad [2.16]$$

Where:

$R_{size}$ = overstrength factor based on the conservative detailing layouts, owing to the restricted sizes and spaces available in construction. An assumed value 1.05 is used for all concrete Seismic Force Resisting Systems (SFRS)

$R_{\Phi}$ = overstrength factor meant to develop the nominal resistance of the member (i.e., $R_{\Phi} = 1/\Phi$). The factor is based on the steel reinforcement because strength design is controlled by the yielding of steel

$R_{yield}$ = overstrength factor based on the difference between the actual yield strength of the material and the one specified. An assumed value of 1.05 is used for all concrete SFRS

$R_{sh}$ = overstrength factor based on the strain hardening characteristics of steel. Given appropriate rebar detailing and concrete confinement, a value of 1.25 is assumed for ductile SFRS cases and 1.10 for moderately ductile ones

$R_{mech}$ = overstrength factor meant to reflect the reserve capacity in the structure before the collapse mechanism is attained. This factor is controlled by the redundancy and sequence of yielding in a structure

For structures designed and detailed according to CSA A23.3-04, NBCC 2005 specifies a $R_o$ value of 1.7 for ductile moment resisting frames and a value of 1.4 for moderately ductile frames. Table 2.6 summarizes the overstrength and ductility factors (Mitchell et al., 2003) for other structural systems:

Table 2.6: $R_o$ and $R_d$ Factors for Concrete Buildings

| Type of SFRS | Ductility | $R_{size}$ | $R_\Phi$ | $R_{yield}$ | $R_{sh}$ | $R_{mech}$ | $R_o$ | $R_d$ |
|---|---|---|---|---|---|---|---|---|
| Moment resisting frames (MRF) | Ductile<br>Moderate Ductility (MD) | 1.05<br>1.05 | 1.18<br>1.18 | 1.05<br>1.05 | 1.25<br>1.10 | 1.05<br>1.00 | 1.7<br>1.4 | 4.0<br>2.5 |
| Coupled Walls<br>Partially coupled walls | Ductile<br>Ductile | 1.05<br>1.05 | 1.18<br>1.18 | 1.05<br>1.05 | 1.25<br>1.25 | 1.05<br>1.05 | 1.7<br>1.7 | 4.0<br>3.5 |
| Shear Walls | Ductile<br>MD | 1.05<br>1.05 | 1.18<br>1.18 | 1.05<br>1.05 | 1.25<br>1.10 | 1.00<br>1.00 | 1.6<br>1.4 | 3.5<br>2.0 |
| MRF/Shear walls with Conventional construction | | 1.05 | 1.18 | 1.05 | 1.00 | 1.00 | 1.3 | 1.5 |

It is interesting to note that the product of the NBCC overstrength and ductility factors for concrete moment resisting frames amount to approximately 8, which is the same value used for response modification coefficient in the 2003 BSSC-FEMA 450 report (BSSC, 2003).

### 2.2.2 Ductility Factor in the 2005 Eurocode

The 2005 Eurocode 8 employs capacity design with the use of the q factor. Bridges are designed to exhibit ductile or limited ductility/essentially elastic behaviour. Ductile structures “should be capable of sustaining at least 5 full cycles of deformation to the ultimate displacement:

- without initiation of failure of the confining reinforcement for reinforced concrete sections, or local buckling effects for steel sections; and
- Without a drop of the resisting force for steel ductile members or without a drop exceeding 20% of the ultimate resisting force for reinforced concrete ductile members.” (BSI, 2005)

Structures with limited ductility need not adhere to the specifications for the ductile structures. Capacity design is therefore not required for such structures because they can be designed to remain elastic. Nonetheless, bridges with limited ductility are assigned q values ranging from 1.0 to1.5. Similarly, the BS EN 1998-1:2004 accounts for overstrength in buildings by specifying a maximum q factor of 1.5 for structures classified as low-dissipative. The BSI ascribes this value to the difference between the design and probable strength of the structure, and the overstrength in structures designed according to capacity design (BSI, 2005; Fardis et al., 2005).

Table 2.7 shows the q factors used in the design of bridges according to the Eurocode 8:

Table 2.7: Eurocode 8 - Response Modification Factors

| **Type of Ductile Members** | **Seismic Behavior** | |
|---|---|---|
| | **Limited Ductile** | **Ductile** |
| Reinforced concrete piers : | | |
| Vertical piers in bending | 1.5 | 3.5 $\lambda$ ($\alpha_s$) |
| Inclined struts in bending | 1.2 | 2.1 $\lambda$ ($\alpha_s$) |
| Steel Piers: | | |
| Vertical piers in bending | 1.5 | 3.5 |
| Inclined struts in bending | 1.2 | 2.0 |
| Piers with normal bracing | 1.5 | 2.5 |
| Piers with eccentric bracing | - | 3.5 |
| Abutments rigidly connected to the deck: | | |
| In general | 1.5 | 1.5 |
| Locked-in structures | 1.0 | 1.0 |
| Arches | 1.2 | 2.0 |

$\alpha_s = L_s/h$ is the shear span ratio of the pier, where $L_s$ is the distance from the plastic hinge to the point of zero moment and h is the depth of the cross-section in the direction of flexure of the plastic hinge.

For $\alpha_s \geq 3$ $\lambda(\alpha_s) = 1.0$

$3 > \alpha_s \geq 1.0$ $\lambda(\alpha_s) = (\alpha_s / 3)^{1/2}$

It is interesting to note that there is no account for structural redundancy in the designation of bridge piers (e.g., single column piers versus multi-column piers). The q factors are therefore conservative for piers with more than one ductile member. Adjustment to the displacement ductility factor for short periods is also included in the Eurocode 8. This adjustment is utilized in the determination of the seismic design displacement.

In line with the capacity design method, the variability in material and strength properties is accounted for by the overstrength factor $\Upsilon_o$. This factor has values of 1.35 and 1.25 for concrete and steel members respectively, and is multiplied by the design flexural strength to get the overstrength moment (BSI, 2005). These design moments are then used to develop the design shear and resistance of the capacity protected members.

### 2.2.3 Capacity Design Approach in the Proposed AASHTO LRFD Provisions

The proposed AASHTO LRFD provisions dictate the use of the displacement based capacity design in designing bridges for earthquakes. The displacement based design is required for structures with ductile concrete substructures, and force based design is used for ductile steel superstructures. The displacement demand is obtained by correlating the elastic displacement with that of the elasto-plastic system, except for bridges with short periods. For short period structures, the provisions provide adjustment factors based on the period of the structure. The displacement demand is found according to the specified analysis procedure as shown in Table 2.8:

Table 2.8: Minimum Seismic Design Analysis Requirements

| Seismic Design Category | Regular Bridges with 2 through 6 spans | Not Regular Bridges with 2 or more Spans |
|---|---|---|
| A | Not Required | Not Required |
| B,C, or D | Equivalent Static or Elastic Dynamic analysis | Elastic Dynamic Analysis |

The displacement capacity is determined empirically for bridges in Seismic Design Categories (SDC) B and C, and using inelastic quasi-static pushover analysis for bridges in SDC D. Special detailing and member proportions is stipulated for each category, to achieve the minimum displacement ductility demand, $\mu_{D,}$ which is determined as follows:

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \qquad [2.17]$$

Where

$\Delta_{pd}$ = plastic displacement demand

$\Delta_{yi}$ = idealized yield displacement corresponding to the idealized yield curvature

Bridges in SDC B or C are designed for a limited ductility response, specifically a $\mu_D$ value not exceeding 4.0. Alternatively, bridges in SDC D are designed for $\mu_D$ values ranging from 4.0 to 6.0.

The provisions dictate the use of capacity design for bridge in SDC C and D only. Plastic hinging forces for reinforced concrete members are determined as follows:

$$M_{po} = \lambda_{mo} M_p \qquad [2.18]$$

Where:

$M_{po}$ = plastic moment capacity of column

$\lambda_{mo}$ = overstrength factor taken as 1.2 and 1.4 for members with ASTM A706 and ASTM A615 Grade 60 reinforcement, respectively

Given the adoption of the more accurate displacement based design for ductile substructures, there is no need to determine the seismic design forces on the energy dissipating elements, and therefore no list of response modification factors exists in the provisions. Response modification factors are provided however for structures with a ductile steel superstructure.

# Chapter 3 Bridge Model and Analysis

## 3.1 Bridge Model in SAP2000

A 2-span regular bridge supported by two abutments and a single column will be analyzed, to compare the investigated design codes quantitatively. The bridge is modelled according to the Standard Drawings for Ramp 427S-407W Overpass (Structural Drawings by Morrison Hershfield Limited Consulting Engineers, 1991), with minor modifications. The original bridge is curved, super-elevated, and has more than 2 spans with piers of varying heights. The author simplified the model to a straight 2-span bridge with constant elevation, supported by one column and two abutments. The structure is constructed with concrete having a minimum compressive strength of 35 MPa, and Grade 400 weldable steel with minimum yield strength of 400 MPa. The specific weight of concrete and steel used is 24 KN/m$^3$ and 77 KN/m$^3$, respectively. The modulus of elasticity of concrete and steel is 26,275 MPa and 200,000 MPa, respectively (CSA, 2006).

The superstructure is a 2,400 mm deep, 13,460 mm wide prestressed concrete box girder with one interior web. The superstructure spans 45 m between vertical supports. The pier is a 2,400 mm circular cast-in-place concrete column, 12 metres high with a clear cover of 80 mm to the transverse reinforcement. The transverse reinforcement is a 15M spiral, which is limited by the CSA A23.3-04 concrete confinement requirements to a 50 mm pitch (CSA, 2004). Preliminary analysis of the bridge resulted in the need for two longitudinal steel configurations as follows:

1. Type A: 36 – 45M bars.
2. Type B: 36 – 55M bars.

The mass source used to generate the seismic loads includes the superstructure dead load, the superimposed dead loads and 1/3 of the column weight. The superimposed dead loads include 90 mm asphalt and waterproofing overlay, and two barriers. The dead load used in the model is 337.5 KN/m, distributed over the length of the bridge, and a 434 KN nodal load placed on the top of the column. The uniform dead load includes an allowance for the weight of light posts, cables and future asphalt overlays. The total dead load acting on the single column is approximately 16,500 KN. This load will be used to develop the nominal flexural resistance of the column using the Response2000 program. Figures 3.1 to 3.4 show the analyzed bridge (Morrison Hershfield Limited Consulting Engineers, 1991) geometry:

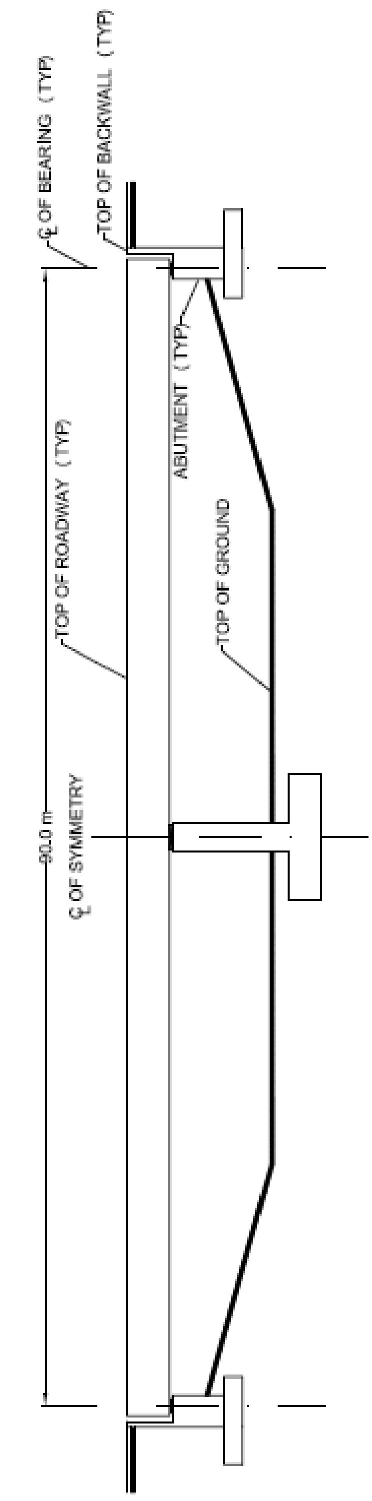



Figure 3.1 Bridge Elevation

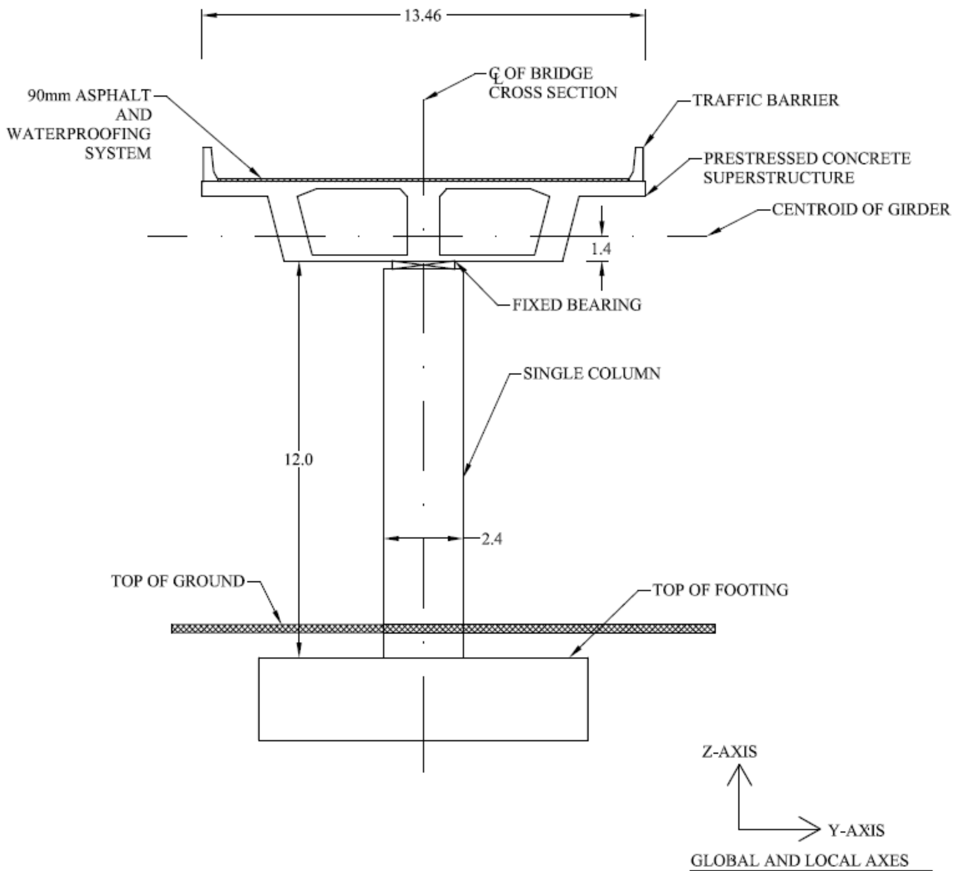



Figure 3.2 Bridge Cross Section

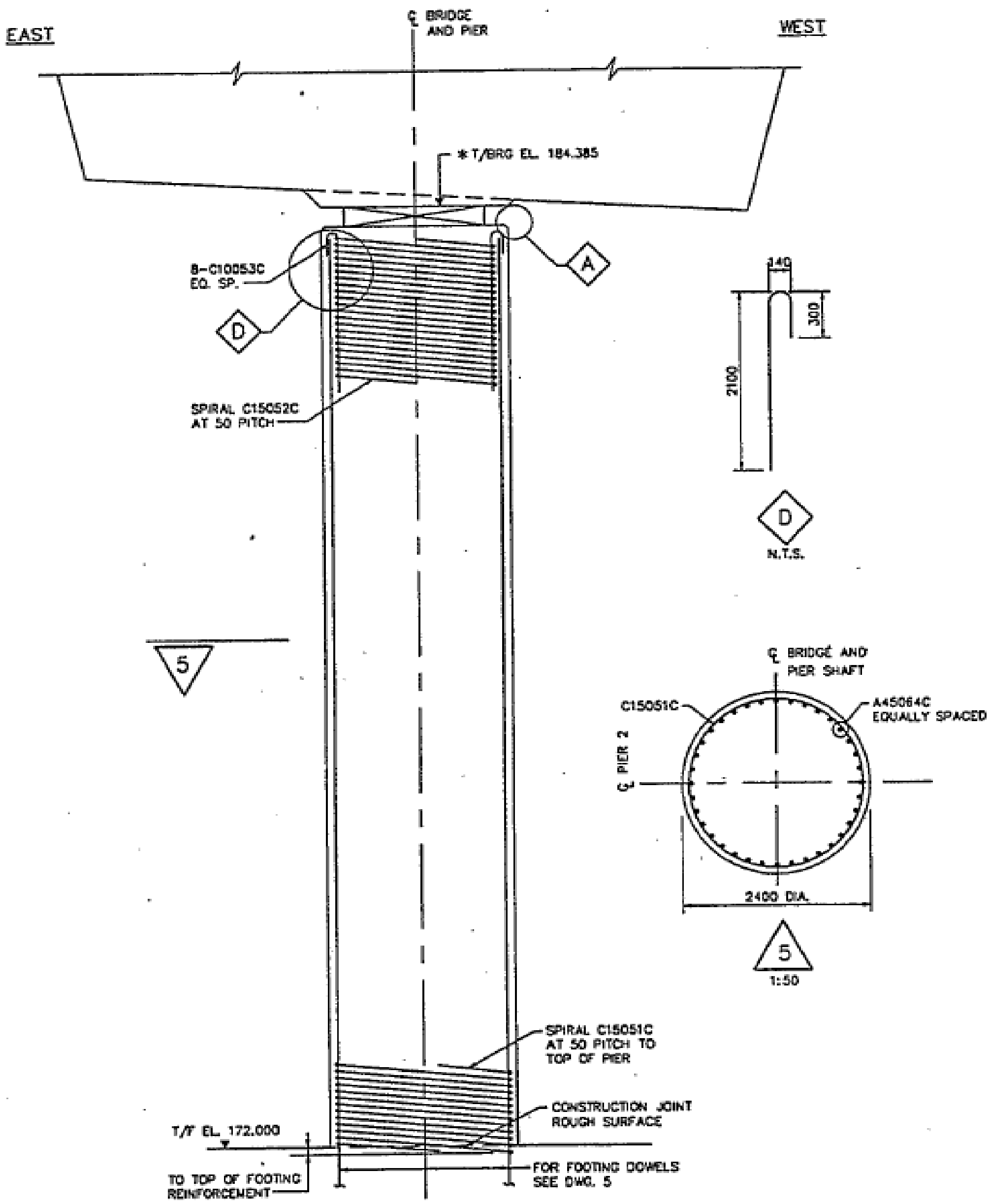



Figure 3.3 Column Details

(Morrison Hershfield Limited Consulting Engineers, 1991)

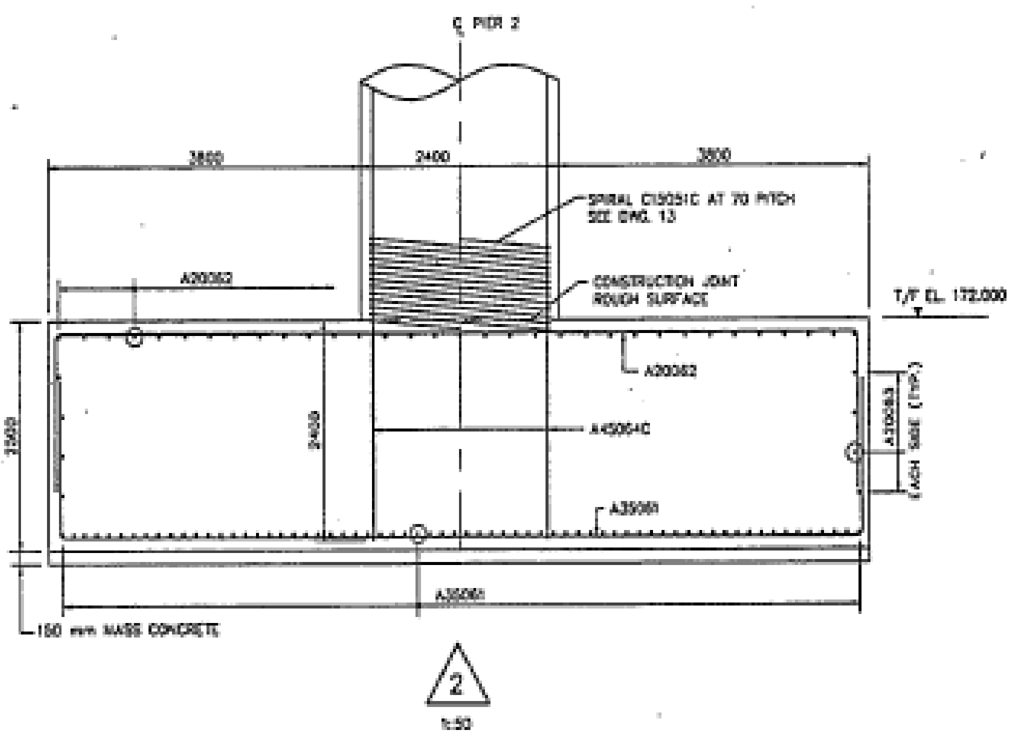



Figure 3.4 Footing Detail

(Morrison Hershfield Limited Consulting Engineers, 1991)

To establish a common basis for comparing the seismic design provisions of the examined codes, the following assumptions are undertaken:

1. The factored and nominal flexural capacity of the concrete column is determined using the Response2000 program (Bentz and Collins, 2000). The column properties and the unfactored dead load on the single column (i.e., 16,500 KN) will be used to develop the flexural capacity.
2. The seismic loads and deflections will be based solely on the effective flexural rigidity of a Type A column.

3. The effect of the vertical seismic loads on the bridge is ignored. The seismic loads in the two orthogonal horizontal directions will be considered only (i.e., along the longitudinal axis of the bridge and transverse to it).
4. The analysis is based on applying each design spectra to the bridge, and comparing the resulting seismic design moments, shears and ductility demands. The goal of this thesis is to compare the effect of the design spectra and load reduction factors on the design loads, for each of the design codes compared.
5. The bridge belongs to the “Other” bridge classification, and has an Importance factor of I = 1.0. This assumption was necessary because it is the only common Importance category in all the codes being studied (e.g. the AASHTO proposed provisions do not address critical/essential bridges specifically). Also, the Eurocode treats the Importance category differently by using different return periods for different Importance categories.
6. The strength analysis of all codes is based on a rare earthquake event.
7. The probability of a high live load during an earthquake is low. The dynamic analysis mass source is based solely on the dead load.
8. The NBCC seismic design spectra will be implemented with a ductility factor of 5.0, and an overstrength factor determined as follows:

$$R_o = R_{size}\ R_{\phi}\ R_{yield}\ R_{sh}\ R_{mech} = 1.05 \times 1.11 \times 1.05 \times 1.25 \times 1.00 = 1.5 \qquad [3.1]$$

Where:

$$R_{\phi} = \frac{1}{\phi_s} = \frac{1}{0.9} = 1.11$$

The results based on the NBCC seismic design spectra will be referred to in this thesis as the “Modified CHBDC” results. A value of $R_{mech}$=1.0 was chosen to reflect the low number of plastic hinges required to form, before an overall bridge collapse is imminent.

9. To overcome the incompatible design philosophy undertaken by the proposed AASHTO LRFD provisions and that of the other codes, the force based capacity design provisions will be assumed. The response modification factor used will be that of the CHBDC. In the case of a single column, this factor equals R = 3.0. This value is below the maximum displacement ductility factor required for SDC B and C of 4.0. In addition, the displacements obtained were compared with the limits specified in the proposed AASHTO guidelines. If the bridge is designated in the Seismic Design Category that requires no seismic design check (i.e., SDC A), then the design provisions for SDC B will be used (Imbsen, 2007).

10. Linear dynamic analysis will be used in all cases, even if an Equivalent Static method maybe applicable, or the seismic design code approach does not require checking the bridge for seismic loads. Vibration modes

will be summed up until the effective modal mass amounts to 90% of the total mass of the bridge.

11. The column is designed to the required detailing for ductile elements as stipulated in all the design codes considered.

Due to the presence of neoprene bearings at the top of the column, the connection between the superstructure and the column is assumed to be simply supported. The column is therefore modelled with a fixed support at its base (i.e., at the footing) and a pinned connection at its top (i.e., at the superstructure). The bearings at the abutments consist of multi-directional and uni-directional sliding bearings, permitting movement in the longitudinal direction of the bridge, with shear keys to resist lateral movements of the bridge. Hence, the earthquake resisting system is based solely on the single column in the longitudinal direction (i.e., along the bridge's longitudinal axis), and on the single column and abutments in the transverse direction (i.e., at 90 degrees to the longitudinal axis of the bridge). The bridge pier will therefore dissipate energy in the longitudinal direction through the action of the concrete column only, and will participate along with the abutments in resisting transverse movements. The inertial load is transferred from the superstructure's centre of gravity to the rock or soil by shear in the abutments and column.

The bridge is modelled in SAP2000 Advanced Version 11 (computers and structures Inc, 2000), as a 90 m long structure pinned at the supports (i.e., rotations are released), with the exception of allowing for longitudinal movement at both abutments. The seismic response spectra are applied independently in each direction. The vibration periods and shapes for each vibration mode is determined by SAP2000, and later summed up using the CQC method. The reinforced concrete column will crack under cyclic seismic loading, and its stiffness will consequently decrease. The decrease in stiffness results in a reduction in the energy dissipating capabilities of the column. Cracked section properties are therefore used to model the reinforced concrete column. Priestley et al. (1996) developed graphs correlating the effective section properties of columns with the axial load and reinforcement ratio as shown in Fig 3.5:

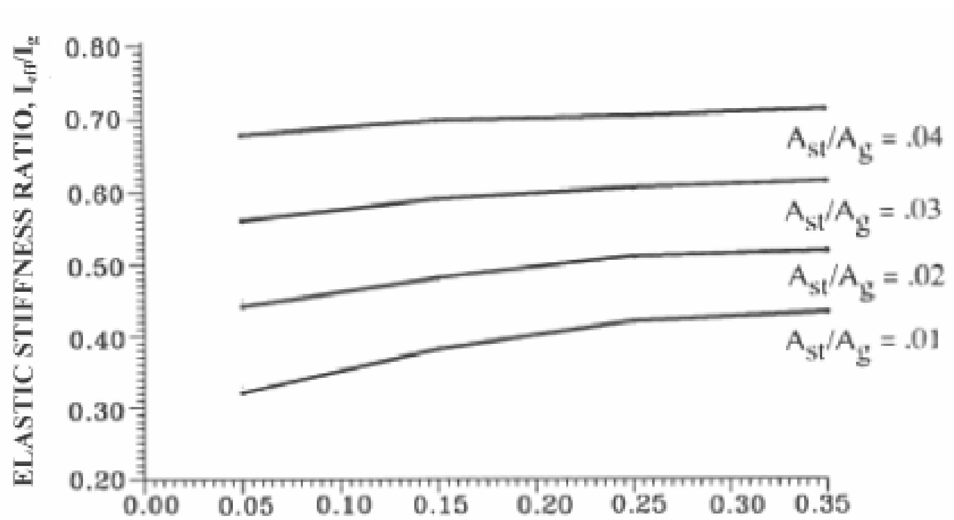



Figure 3.5 Effective Section Properties for Circular Columns

The ratio of the longitudinal steel reinforcement to the gross area, for a Type A column is:

$$\frac{A_{st}}{A_g} = \frac{36 \times 1500 mm^2}{\frac{\pi \phi^2}{4}} = 0.01 \qquad [3.2]$$

Where

$A_g$ = gross area of the column, equal to 4,523,893 $mm^2$ for a column with diameter $\Phi$ = 2400 mm

The ratio of the axial load to the axial load capacity for a Type A column is:

$$\frac{P}{f'_c \times A_g} = \frac{16500 \times 10^3 \ N}{35 \ MPa \times 4523893} = 0.1 \qquad [3.3]$$

The ratio of the effective section property to the gross section property, using the above values and graph, is 0.36. This value is inserted into the section modification factors in SAP2000, for all three principal axes. The superstructure (i.e., the capacity protected member) will remain elastic, and will be modelled with its full section properties.

To represent the highly populated regions in Canada with different seismic ground motions, the bridge will be analyzed for the city centres of Montreal, Vancouver and Toronto seismic ground motions. The NEHRP Site Class B and E will be used for the three cities considered. Spectral accelerations for an event with a 1000 year return period were obtained from the Natural Resources of Canada website (http://earthquakescanada.nrcan.gc.ca/hazard/interpolator/index_e.php), to establish the design spectra per the proposed AASHTO provisions. The elastic coefficient spectra incorporated into SAP2000 are as shown below:

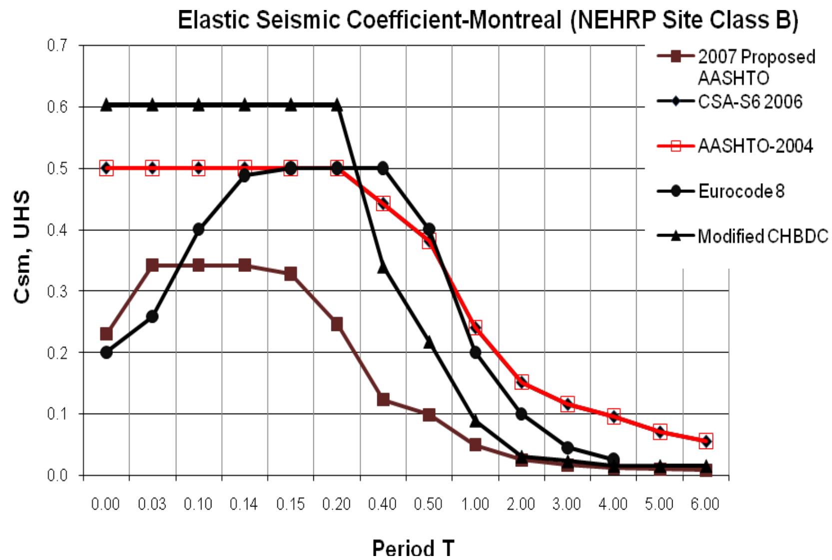



Figure 3.6 Elastic Seismic Coefficients for Montreal-NEHRP Site Class B

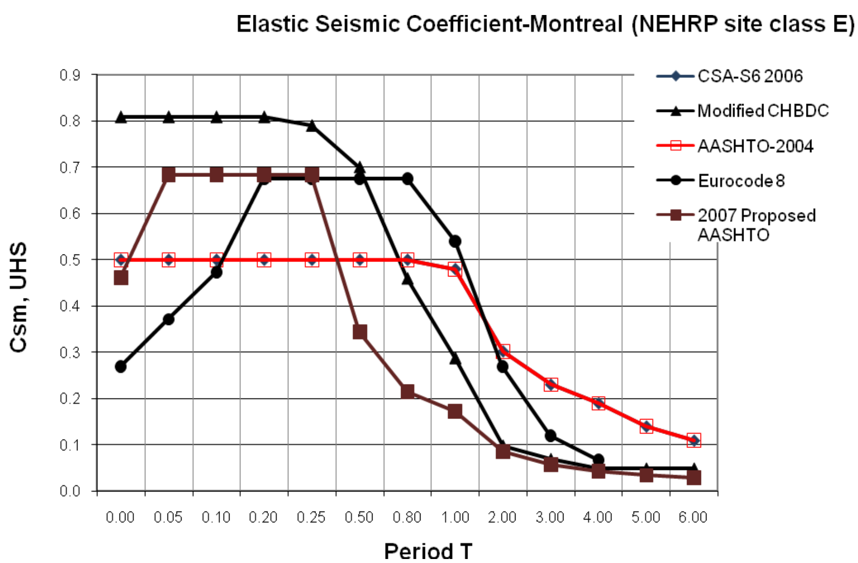



Figure 3.7 Elastic Seismic Coefficients for Montreal-NEHRP Site Class E

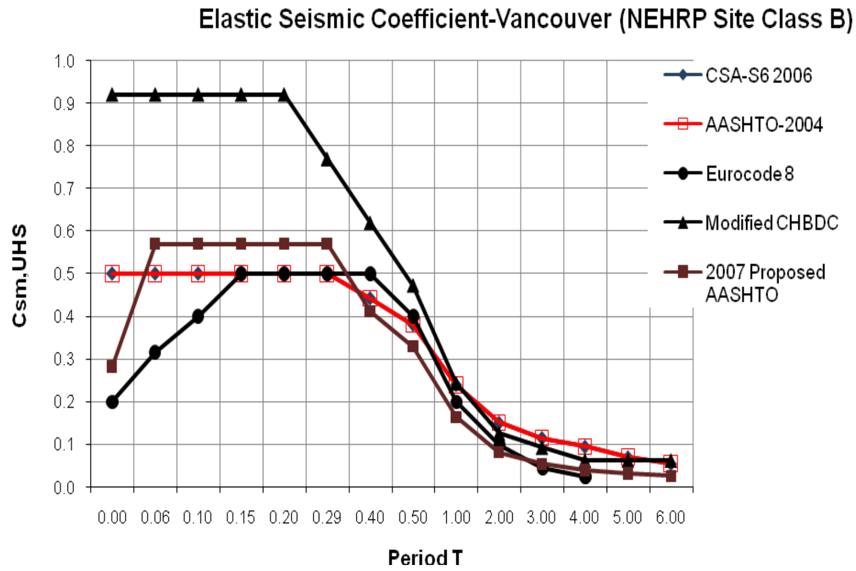



Figure 3.8 Elastic Seismic Coefficients for Vancouver-NEHRP Site Class B

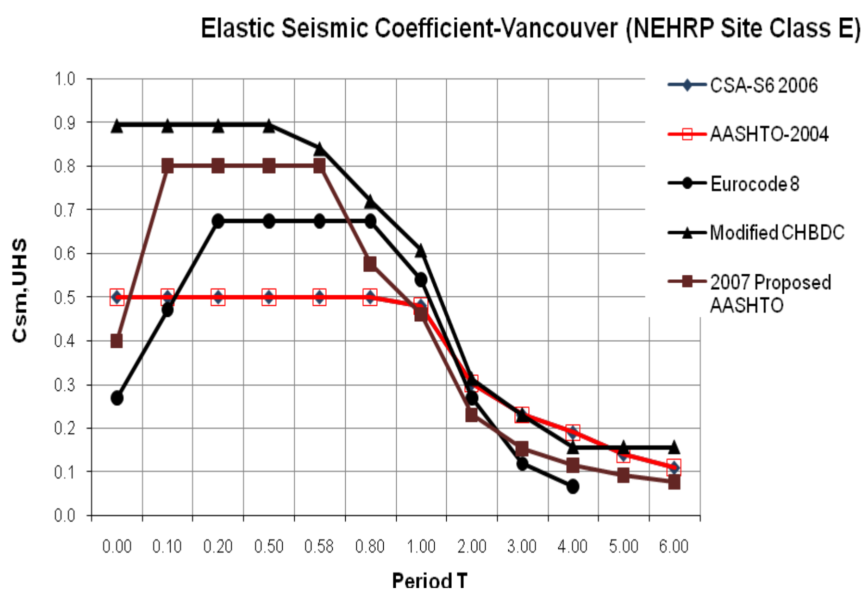



Figure 3.9 Elastic Seismic Coefficients for Vancouver-NEHRP Site Class E

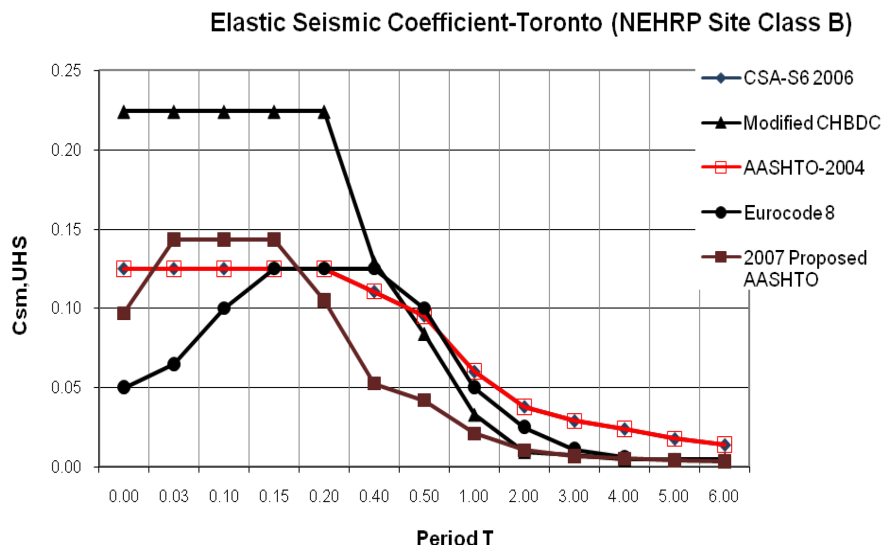



Figure 3.10 Elastic Seismic Coefficients for Toronto-Site Class B

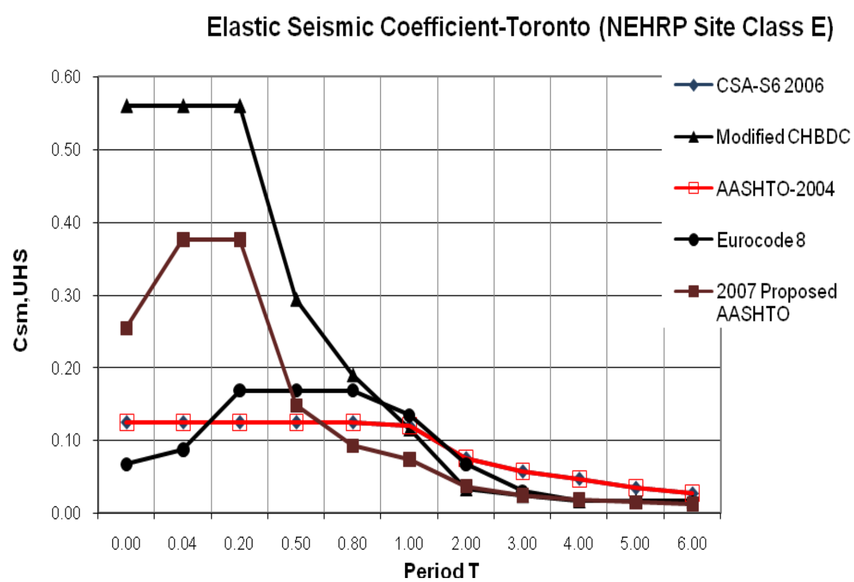



Figure 3.11 Elastic Seismic coefficients for Toronto-Site Class E

SAP2000 version 11 was used to perform the linear dynamic analysis of the bridge. The bridge superstructure was modelled as a line element, subdivided into four elements. The seismic event is translated into a dynamic load on the bridge using the mass source defined above. In SAP2000, the inserted mass of the structure is lumped at the node. Four nodes within each span are considered sufficient given the simple geometry of the bridge (MCEER/ATC, 2003).

The reinforced concrete column is modelled as one segment, with the effective section properties determined by multiplying the gross section properties by 0.36. The abutments and footing are modelled as rigid supports (i.e., nodes). The SAP 2000 bridge frame elements run parallel to the centroid of the bridge (i.e., at 1.4 m from the bottom of the superstructure) as shown in Fig. 3.12:

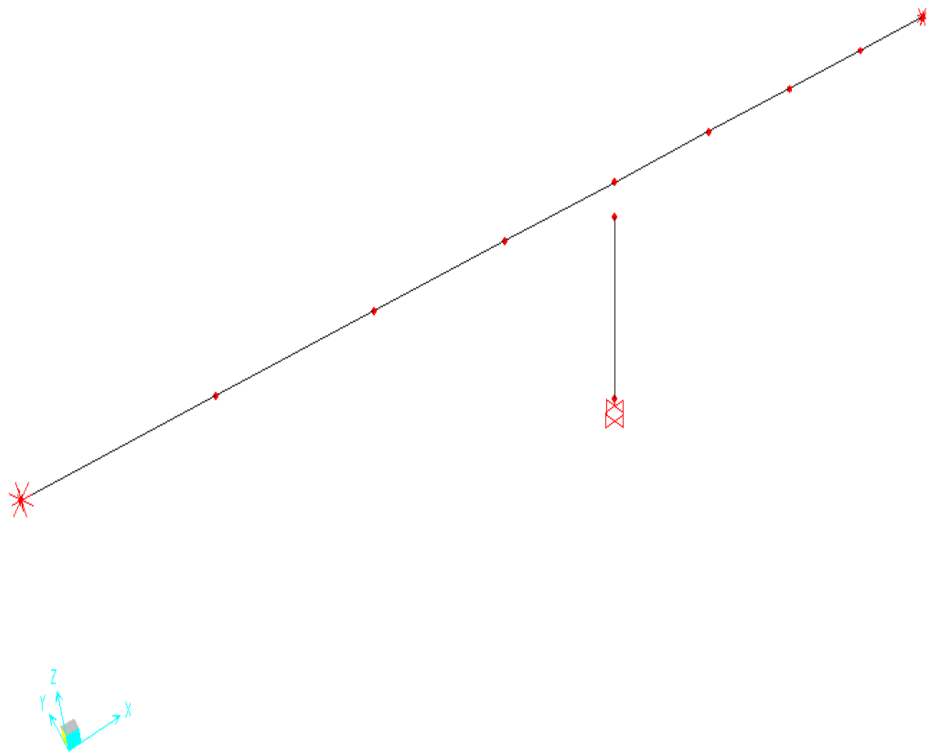



Figure 3.12 Bridge Model in SAP2000

The section properties used in the SAP2000 model are shown in Table 3.1:

Table 3.1: Section Properties used in the SAP2000 Bridge Model

(refer to Fig. 3.2 for local axes orientations)

| **Section Property** | **Prestressed Concrete Box Girder** | **Reinforced Concrete Column** |
|---|---|---|
| Cross Sectional Area ($m^2$) | 11.1 | 4.5 |
| Centroidal moment of Inertia about the local Y-Axis $I_y$ ($m^4$) | 7.64 | 1.63 |
| Centroidal moment of Inertia about the local Z-Axis $I_z$ ($m^4$) | 147 | 1.63 |
| Torsional Constant - $I_x$ ($m^4$) | 154.6 | 3.25 |
| Weight (KN) | 23,900 | 1,302 |
| Location of centroid w.r.t the local Z- axis (m) | 1.4 From bottom of Girder | |
| Shear Area in both local axes ($m^2$) | 4.7 | 4.07 |
| Section Modulus-$S_y$ ($m^3$) | 5.4 | 1.36 |
| Section Modulus-$S_z$ ($m^2$) | 21.84 | 1.36 |
| Radius of Gyration -$r_y$ (m) | 0.83 | 0.6 |
| Radius of Gyration-$r_z$ (m) | 3.64 | 0.6 |

# Chapter 4 Results and Discussion

## 4.1 Results

### 4.1.1 Overview of Design Loads and Column Capacities Development

The SAP2000 output includes the flexural and shear loads in the frame elements, and the displacements of the nodes defining these elements. The reported period at the fundamental vibration mode is 2.2 seconds. It is noted that the fundamental vibration modes in each of the horizontal directions have mass participating factors greater than 90%. This suggests that an Equivalent Static method may be used in lieu of the linear dynamic analysis method, and that the primary response is represented by the fundamental modes of vibration.

The results are specified for each design spectrum, and represent absolute values in each of the principal horizontal directions. The reported moments, shears and deflections are combined as follows:

$$F_1 = \sqrt{\left[(F_x)^2 + (0.3 \times F_y)^2\right]} \qquad [4.1]$$

$$F_2 = \sqrt{\left[(0.3 \times F_x)^2 + (F_y)^2\right]} \qquad [4.2]$$

Where:

$F_1$, $F_2$ = Resultant elastic moment, elastic shear or deflection

$F_x$ = Elastic moment, elastic shear or deflection in the global X-direction (i.e., along the bridge's longitudinal axis)

$F_y$ = Elastic moment, elastic shear or deflection in the global Y-direction (i.e., perpendicular to the bridge's longitudinal axis)

The nominal and factored flexural resistance, and the ultimate curvature of the columns used were developed using the Response2000 program. The parameters used to develop these capacities were the unfactored dead load of 16,500 KN, and material resistance factors of 0.65 and 0.85 for concrete and steel, respectively (CSA, 2004). The factored shear resistance was developed according to the CSA A23.3-04 provisions with β = 0.1 and θ=45°. The following equations from the CSA A23.3-04 shear provisions were used.

$$V_c = \phi_c \lambda \beta \sqrt{f'_c}\, b_w d_v \qquad [4.3]$$

$$V_s = \frac{\phi_s\, A_v\, f_y\, d_v \cot\theta}{s} \qquad [4.4]$$

Where

$V_c$ = factored shear resistance attributed to the concrete

$\Phi_c$ = resistance factor for concrete

$\lambda$ = factor to account for low-density concrete

$\beta$ = factor accounting for shear resistance of cracked concrete

$f'_c$ = specified compressive strength of concrete

$b_w$ = effective web width/diameter of circular section

$d_v$ = effective shear depth, taken as the greater of 0.72h or 0.9d

$V_s$ = factored shear resistance attributed to steel

$\Phi_s$ = resistance factor for non-prestressed reinforcing bars

$A_v$ = area of shear reinforcement within a distance s. For spirally reinforced columns, twice the cross sectional area of the transverse steel is used

$f_y$ = specified yield strength of non-prestressed reinforcement

$\theta$ = angle of inclination of diagonal compressive stresses to the longitudinal axis of the member

s = spacing of shear reinforcement measured parallel to the longitudinal axis of the member

Computations based on the unconfined and confined concrete properties were used to determine the column response and ductility. The results exhibit the effect of confining concrete in improving the strength and ductility of the column. The parameters that influence the results include the cross sectional area and spacing of the transverse reinforcement, spatial distribution of the longitudinal reinforcement, the concrete strength and the reinforcement yield strength. Appendix A includes equations that incorporate the aforementioned parameters to define the stress-strain relationship of the confined column section (Légeron and Paultre, 2003). The equations used to develop the displacement ductility of the single column can be found in Appendix A. Figures 4.1 and 4.2 show the compressive stress-strain responses for the unconfined and confined concrete, as well as the resulting moment-curvature relationship, respectively:

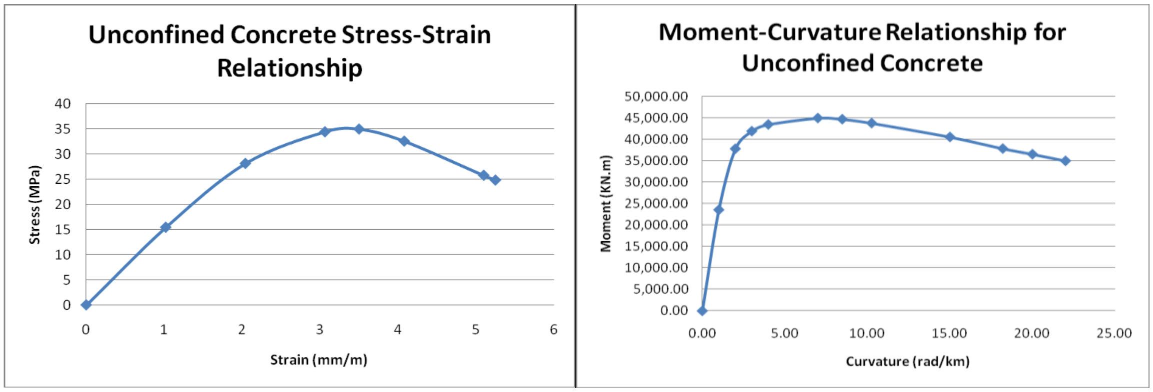



Figure 4.1 Stress-Strain & Moment-Curvature Graphs-Unconfined Concrete Properties

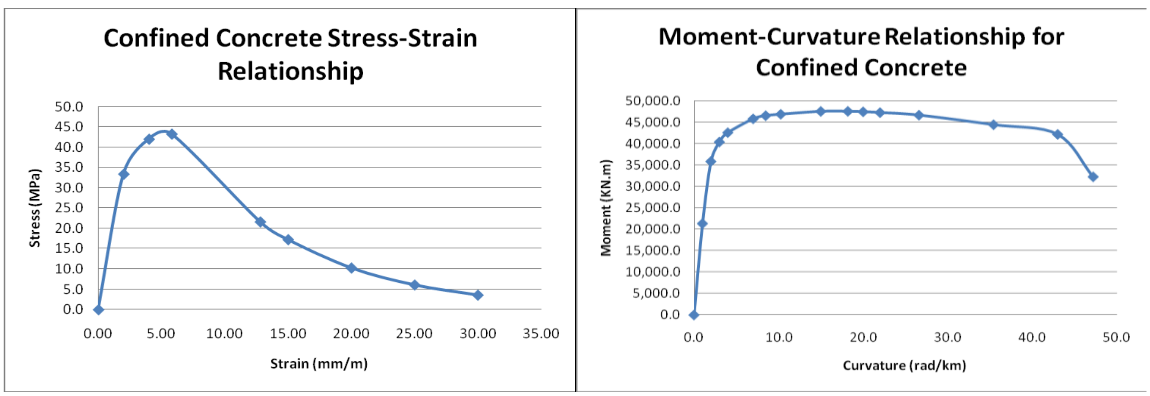



Figure 4.2 Stress-Strain & Moment-Curvature Graphs-Confined Concrete Properties

A summary of the column capacities used in this thesis are presented in Table 4.1:

Table 4.1: Capacity of the Concrete Columns Used

| Column Type | Steel Reinforcement Configuration | Factored Flexural Resistance – $M_r$ (KNm) | Nominal Flexural Resistance - $M_n$ (KNm) | Factored Shear Resistance (KN) | Column Displacement Ductility-Unconfined Concrete | Column Displacement Ductility-Confined Concrete |
|---|---|---|---|---|---|---|
| A | 36 – 45M | 27,945 | 33,665 | 6,295 | 4.65 | 8.74 |
| B | 36 – 55M | 36,589 | 44,475 | 6,295 | 4.65 | 8.74 |

Column Type A was used in all cases except when the design moment in the column exceeded its factored flexural resistance (e.g., when the bridge is designed for the Montreal and Vancouver NEHRP Soil Class E ground motions, using the CHBDC and AASHTO) .

### 4.1.2 Design Moments

The design moment is obtained by reducing the maximum moment by the ductility and the overstrength factors (where applicable) as follows:

$$M_{design} = \frac{\max\ (F_1, F_2)}{R_d R_o} \qquad [4.5]$$

The ductility and overstrength factors employed in this thesis are shown in Table 4.2:

Table 4.2: Factors for a Single Ductile Column

| **Design Code** | **Ductility Factor** $R_d$ | **Overstrength Factor** $R_o$ |
|---|---|---|
| CHBDC (2006) | 3 | N/A |
| AASHTO (2004) | 3 | N/A |
| Modified CHBDC | 5 | 1.5 |
| Eurocode 8 (2005) | 3.5 | N/A |
| Prop. AASHTO provisions | 3 | N/A |

The modified CHBDC values given in Table 4.2 reflect the best estimate of the ductility-related response modification factor, and the overstrength related response modification factor for a single ductile column. These values will be used with the seismic loads developed from the NBCC design spectrum. Figures 4.3 to 4.7 represent the design moments at the base of the bridge column in Montreal, Toronto and Vancouver, for NEHRP Soil Classes B and E. Included in the Figures are the factored flexural resistances.

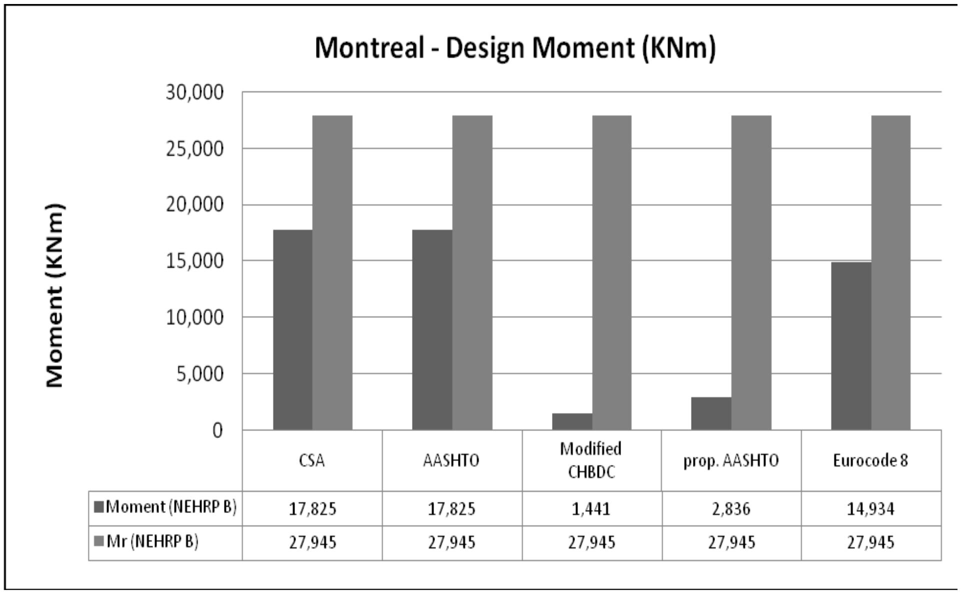



Figure 4.3 Design Moments of Bridge Column in NEHRP Soil Class B - Montreal

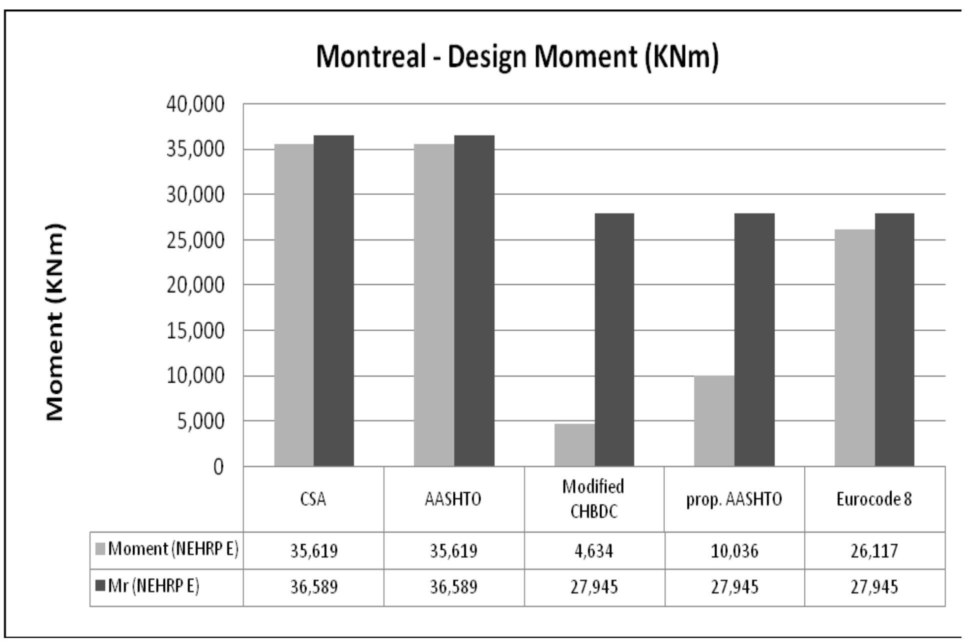
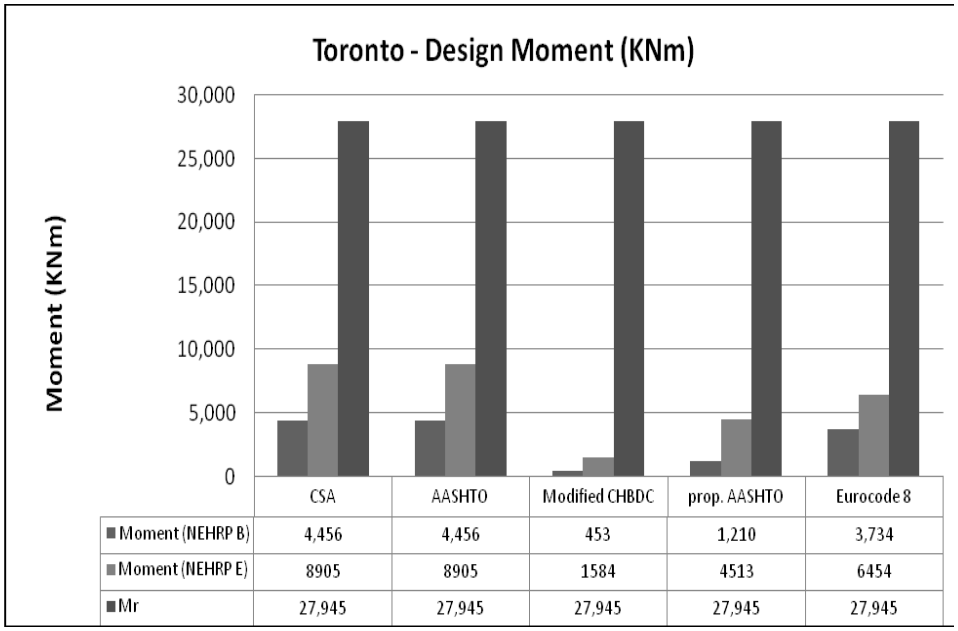
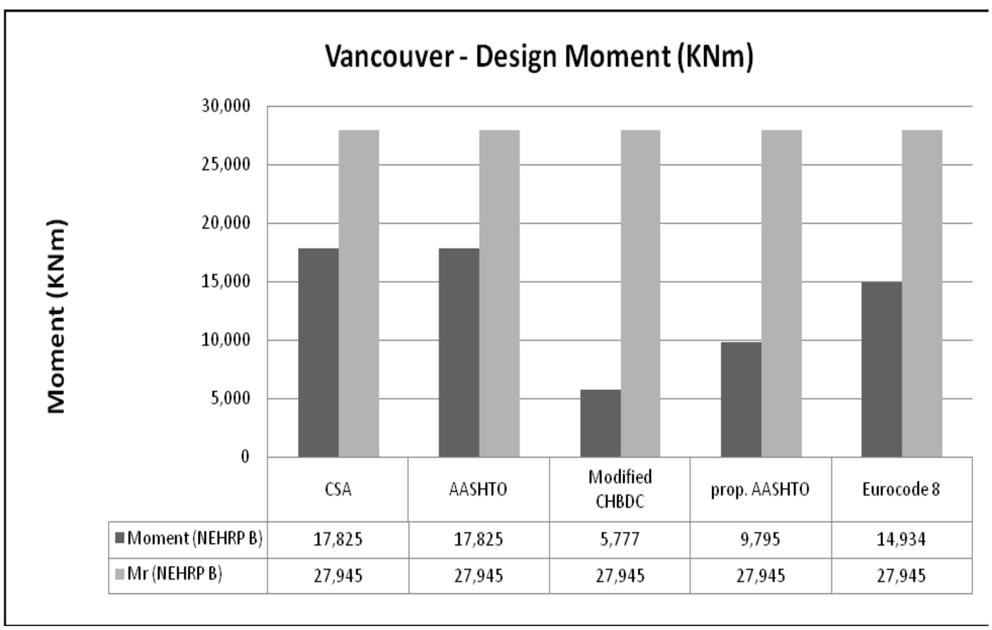



Figure 4.4 Design Moments of Bridge Column in NEHRP Soil Class E - Montreal




| | CSA | AASHTO | Modified CHBDC | prop. AASHTO | Eurocode 8 |
|---|---|---|---|---|---|
| Moment (NEHRP B) | 4,456 | 4,456 | 453 | 1,210 | 3,734 |
| Moment (NEHRP E) | 8905 | 8905 | 1584 | 4513 | 6454 |
| Mr | 27,945 | 27,945 | 27,945 | 27,945 | 27,945 |

Figure 4.5 Design Moments of Bridge Column in NEHRP Soil Classes B&E - Toronto




| | CSA | AASHTO | Modified CHBDC | prop. AASHTO | Eurocode 8 |
|---|---|---|---|---|---|
| Moment (NEHRP B) | 17,825 | 17,825 | 5,777 | 9,795 | 14,934 |
| Mr (NEHRP B) | 27,945 | 27,945 | 27,945 | 27,945 | 27,945 |

Figure 4.6 Design Moments of Bridge Column in NEHRP Soil Class B - Vancouver

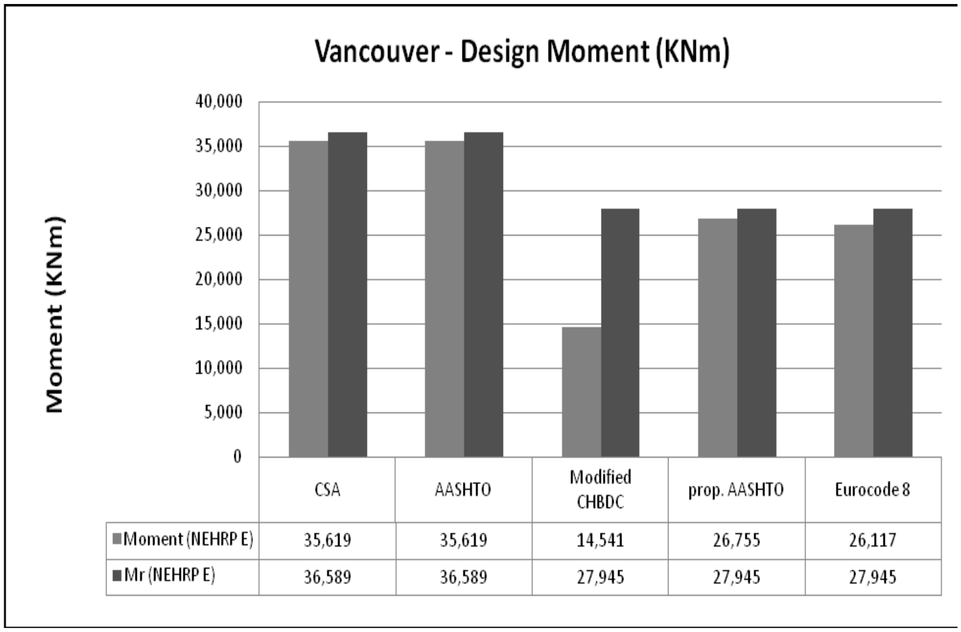



Figure 4.7 Design Moments of Bridge Column in NEHRP Soil Class E - Vancouver

### 4.1.3 Design Shear Loads

The design shear load is the minimum of the elastic shear load (i.e., with $R_d$ = 1.0 and $R_o$ = 1.0), and the shear resulting from the inelastic hinging moments. As mentioned earlier, this force is developed from the probable flexural resistance, by multiplying the nominal moment-$M_n$ by a factor specified in the codes. Figure 4.8 is a graphical presentation of the procedure used to develop the shear load resulting from the inelastic hinging moments of the column (Imbsen, 2007):

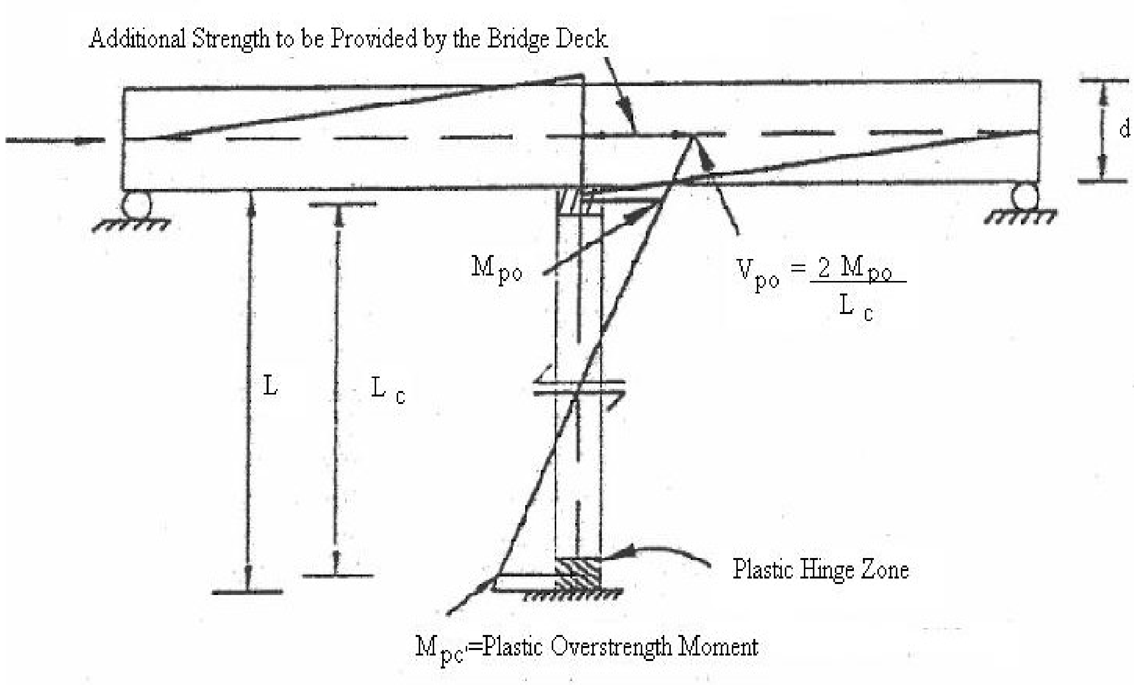



Figure 4.8 Inelastic Hinging Forces (Imbsen, 2007)

The analyzed bridge differs from the represented figure in that the column is a cantilever, whereby the value of $L_c$ is the difference between half the plastic hinge length and the clear height of the column. The difference arises because the cantilever column bends in single curvature.  In addition, the moment at the top of the column is nil. The shear load based on the inelastic hinging moments is developed as follows:

$$V_{design} = \frac{M_{top} + M_{bottom}}{L_c} \qquad [4.6]$$

Where:

$V_{design}$ = shear load based on the inelastic hinging moments, KN

$M_{top}$ = probable flexural resistance at the top of the column, KN.m. The value of $M_{top}$ for the analyzed bridge is nil

$M_{bottom}$ = probable flexural resistance at the base of the column, KN.m

$L_c$ = design shear span, equal to H – (0.5$L_p$)

H = is the clear height of the column and is equal to 12 m (i.e., 39.36 ft.)

The plastic hinge length - $L_{p,}$ is determined (Imbsen, 2007; NCHRP 12-49) as follows:

$$L_p = (0.08H) + (4400\varepsilon_y d_b) \qquad \text{(feet)} \qquad [4.7]$$

$$L_p = (0.08 \times 39.36\ ft) + (4400 \times 0.0021 \times 0.143\ ft) = 4.47 ft. \cong 1.36\ m$$

Where:

$L_p$ = length of the plastic hinge in the column

$d_b$ = diameter of the longitudinal steel reinforcement

$\varepsilon_y$ = yield strain of the longitudinal reinforcement

The nominal flexural resistance $M_n$ of the column types used are presented in Table 4.1. The probable flexural resistance is obtained by multiplying the nominal flexural resistance $M_n$ by the overstrength factors. The overstrength factors employed in this thesis are shown in Table 4.3:

Table 4.3: Factors used to develop the Probable Moments

| **Design Code** | CHBDC (2006) | AASHTO (2004) | Modified CHBDC (2005) | Eurocode 8 (2005) | Proposed AASHTO provisions (2007) |
|---|---|---|---|---|---|
| **Overstrength Factor, γo** | 1.30 | 1.30 | 1.25 | 1.35 | 1.40 |

Figures 4.9, 4.10 and 4.11 represent the design shear loads of the bridge column in Montreal, Toronto and Vancouver respectively, for NEHRP Soil Classes B and E. Included in the Figures are the factored shear resistances.

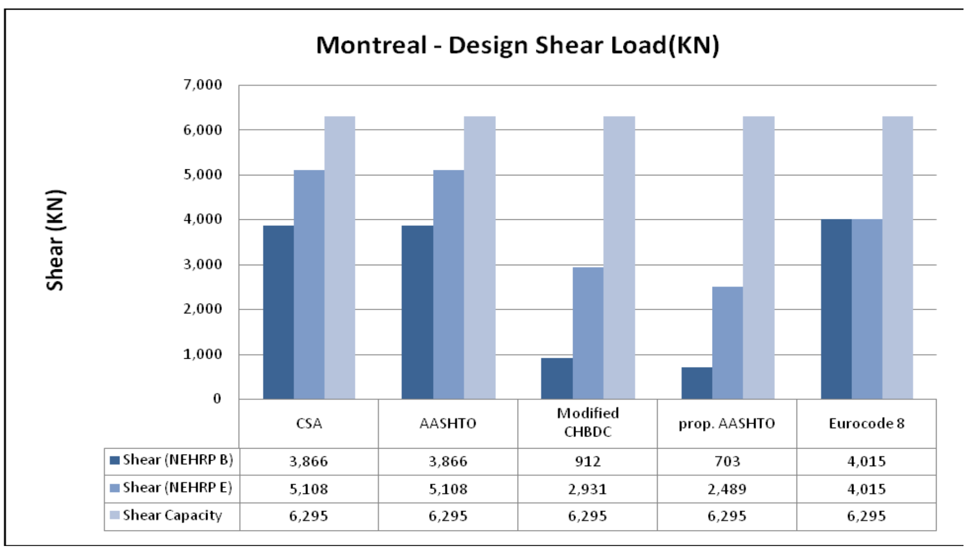
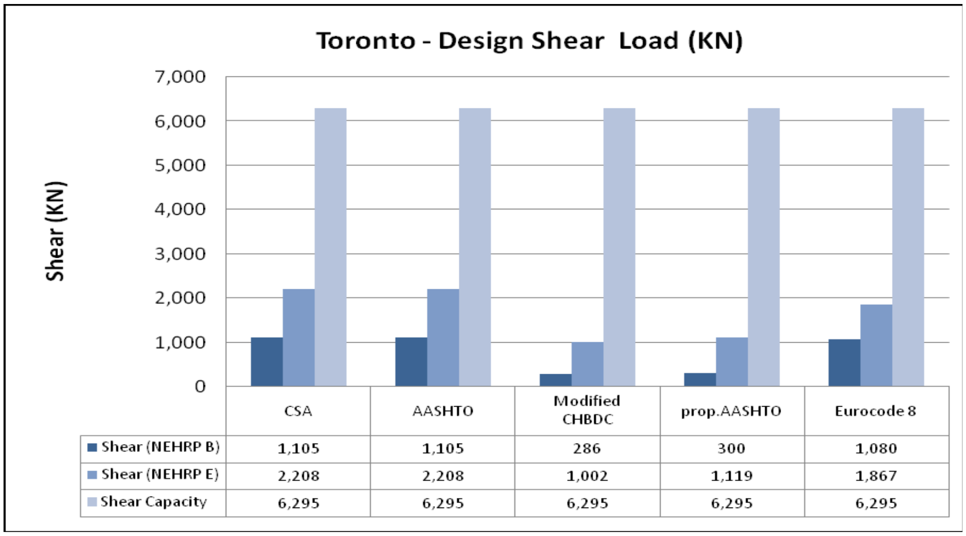
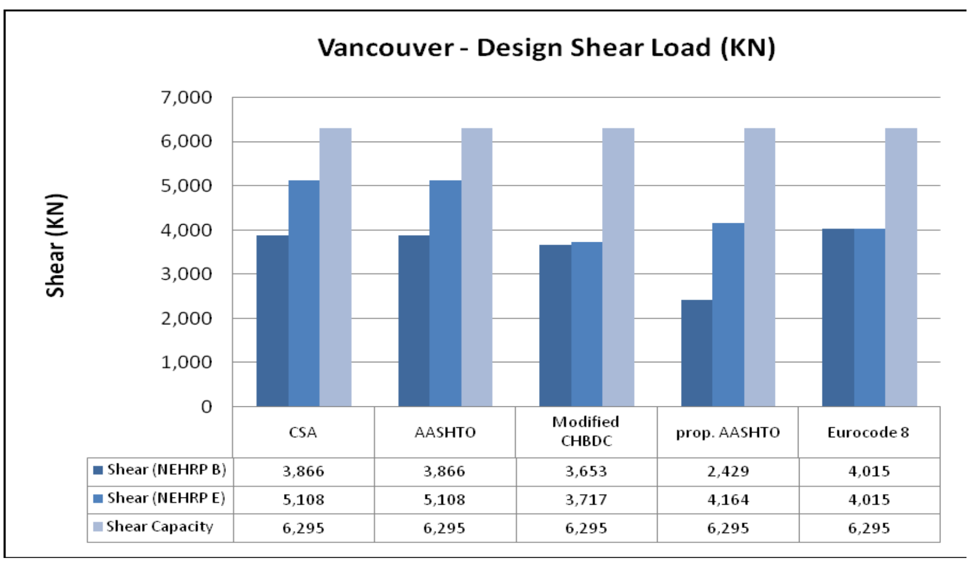



Figure 4.9 Design Shear Loads of Column in NEHRP Soil Classes B&E - Montreal




| | CSA | AASHTO | Modified CHBDC | prop.AASHTO | Eurocode 8 |
|---|---|---|---|---|---|
| Shear (NEHRP B) | 1,105 | 1,105 | 286 | 300 | 1,080 |
| Shear (NEHRP E) | 2,208 | 2,208 | 1,002 | 1,119 | 1,867 |
| Shear Capacity | 6,295 | 6,295 | 6,295 | 6,295 | 6,295 |

Figure 4.10 Design Shear Loads of Column in NEHRP Soil Classes B&E - Toronto




| | CSA | AASHTO | Modified CHBDC | prop. AASHTO | Eurocode 8 |
|---|---|---|---|---|---|
| Shear (NEHRP B) | 3,866 | 3,866 | 3,653 | 2,429 | 4,015 |
| Shear (NEHRP E) | 5,108 | 5,108 | 3,717 | 4,164 | 4,015 |
| Shear Capacity | 6,295 | 6,295 | 6,295 | 6,295 | 6,295 |

Figure 4.11 Design Shear Loads of Column in NEHRP Soil Classes B&E - Vancouver

### 4.1.4 Seismic Deflections and Ductility Demand

The Seismic Ductility Demand ($\mu_D$) is determined (Priestley et al., 2007) as follows:

$$\mu_D = \frac{\Delta_{max}}{\Delta_y} \qquad [4.8]$$

$$\Delta_y = \frac{\phi_y H^2}{3} \qquad [4.9]$$

$$\phi_y = \frac{2.25\varepsilon_y}{D} \qquad [4.10]$$

Where:

$\mu_D$ = Seismic Ductility Demand

$\Delta_{max}$ = the resultant displacement, determined according to Equations 4.1 and 4.2

$\Delta_y$ = the displacement at yield, determined from Equations 4.9 and 4.10

$\Phi_y$ = column curvature at yield

H = clear height of the column

$\varepsilon_y$ = strain in steel at yield

D = column diameter

Figures 4.12 to 4.16 compare the seismic ductility demand with the displacement ductility of the investigated bridge columns in Montreal, Toronto and Vancouver, for NEHRP Soil Classes B and E.

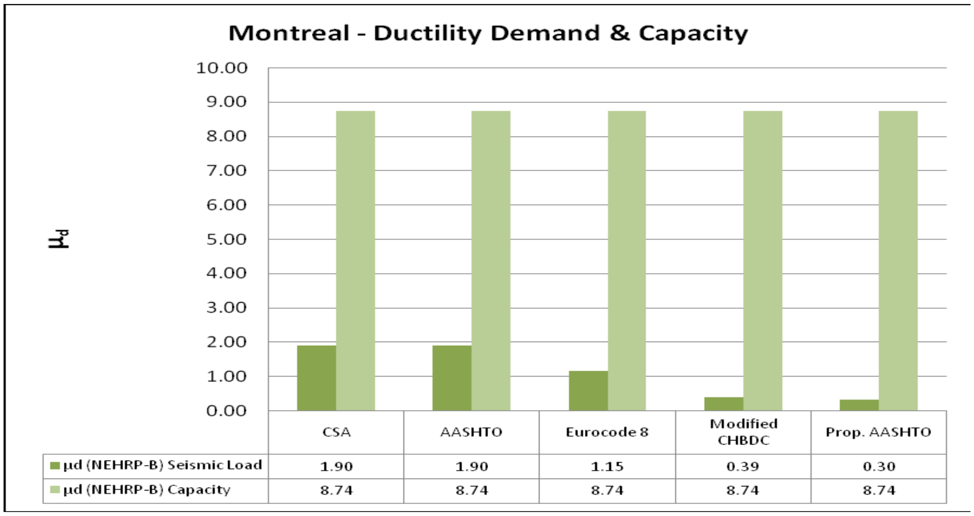



Figure 4.12 Bridge Ductility Demand in NEHRP Soil Class B - Montreal

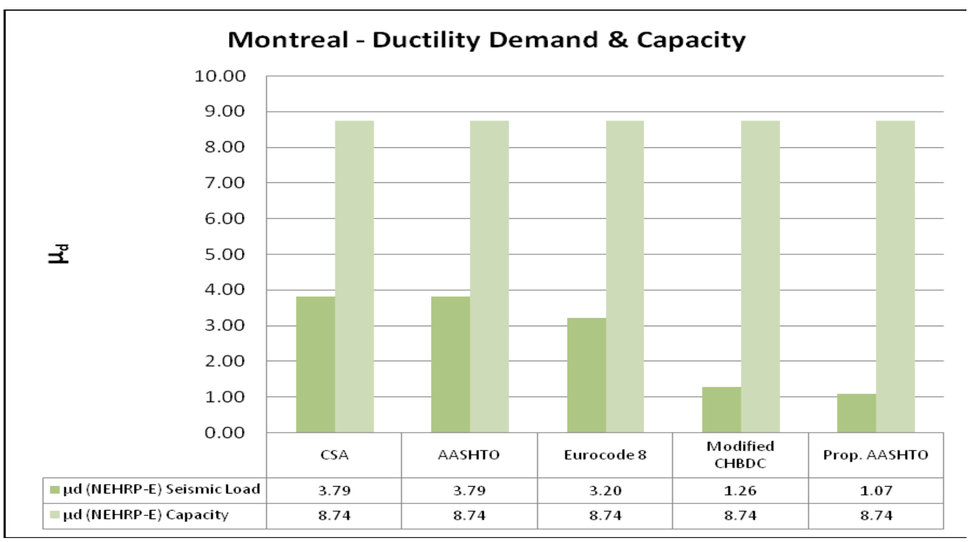



Figure 4.13 Bridge Ductility Demand in NEHRP Soil Class E - Montreal

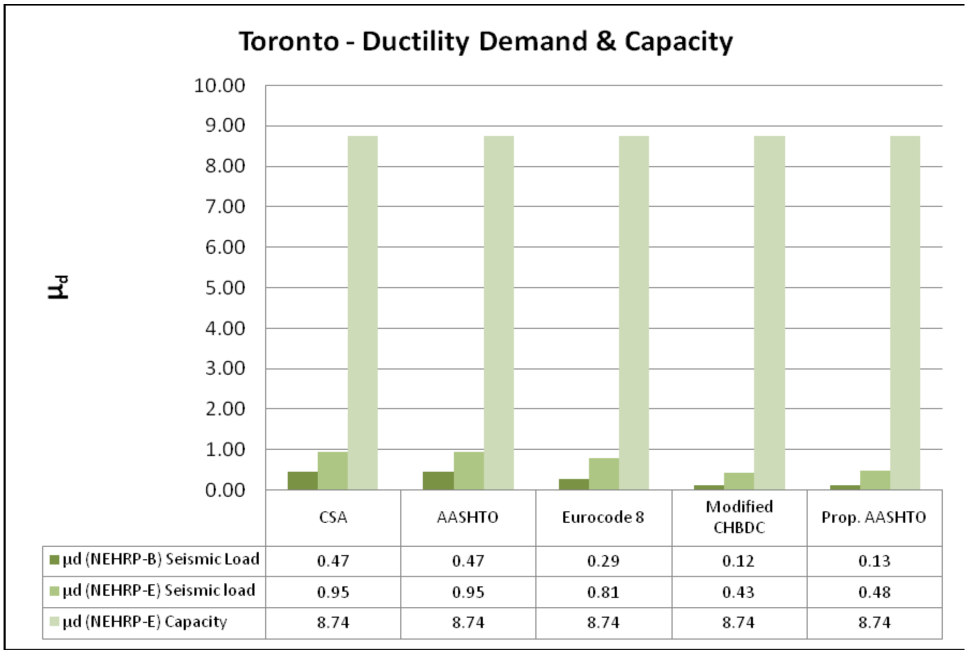



Figure 4.14 Bridge Ductility Demand in NEHRP Soil Classes B&E – Toronto

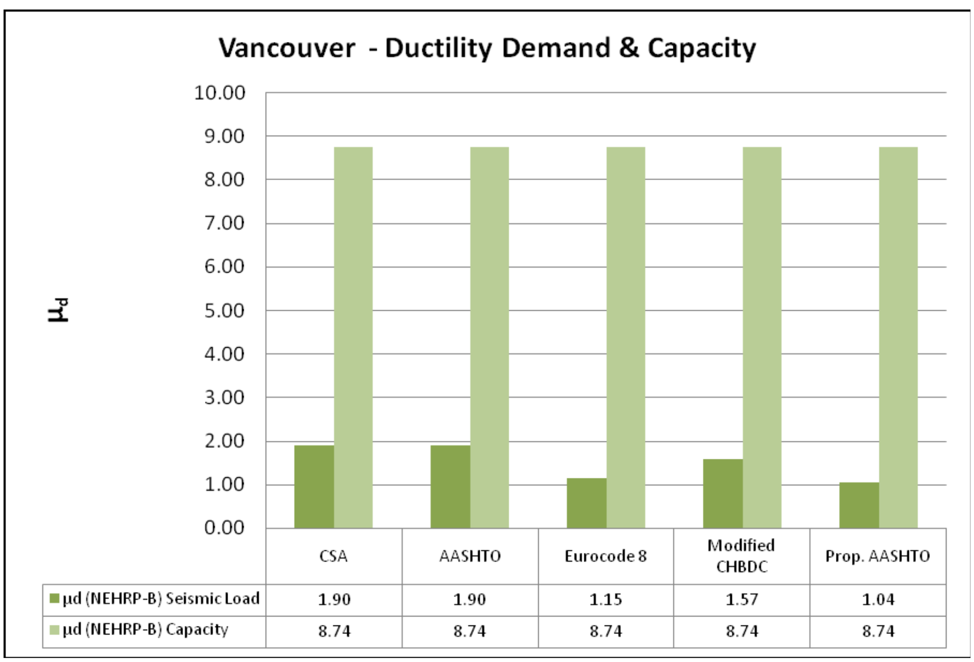
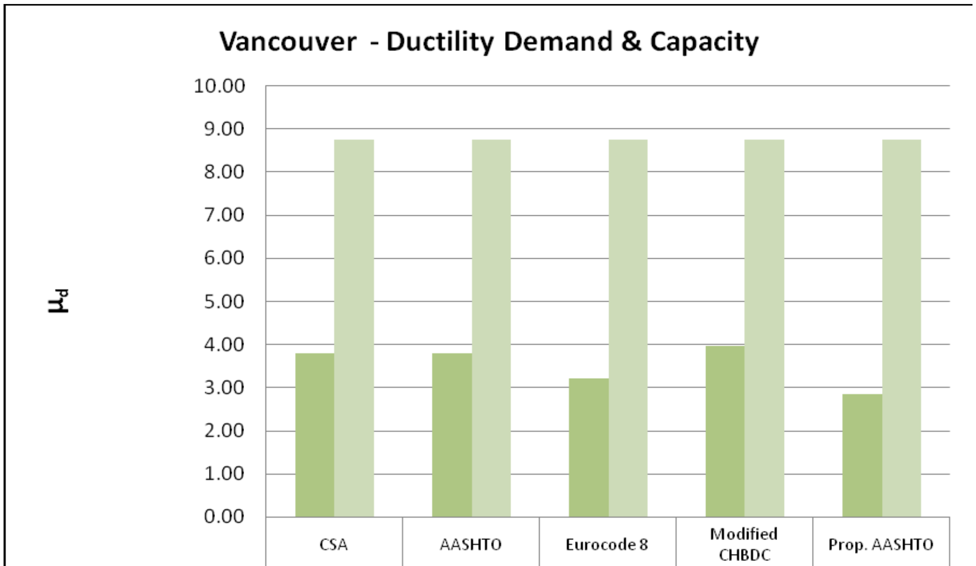



Figure 4.15 Bridge Ductility Demand in NEHRP Soil Class B – Vancouver




| | CSA | AASHTO | Eurocode 8 | Modified CHBDC | Prop. AASHTO |
|---|---|---|---|---|---|
| μd (NEHRP-E) Seismic Load | 3.79 | 3.79 | 3.20 | 3.95 | 2.85 |
| μd (NEHRP-E) Capacity | 8.74 | 8.74 | 8.74 | 8.74 | 8.74 |

Figure 4.16 Bridge Ductility Demand in NEHRP Soil Class E – Vancouver

### 4.1.5 Seismic Design Base Shear per unit Weight of the Bridge

Graphs of the seismic base shear per unit weight of the bridge for the investigated cities are provided in this section. The graphs were provided to verify the results presented above and to examine the seismic design loads at various period ranges. The graphs were developed as follows:

$$\frac{V}{W} = \frac{(C_{sm}, S(T))}{R} \qquad [4.11]$$

Where:

V/W = the seismic base shear per unit weight

$C_{sw}$ = the elastic seismic response coefficient for the code being evaluated

S(T) = the design spectral acceleration at period T for the code being evaluated

R = response modification factor for the code being evaluated. The value is either the product of the ductility and overstrength factors or the response modification factor only

The Importance factor and the NBCC factor, $M_v$, used to account for higher mode effects are taken as one. Figures 4.17, 4.18 and 4.19 contain the design base shear per unit weight for Montreal, Vancouver and Toronto respectively, for NEHRP Soil Class B.

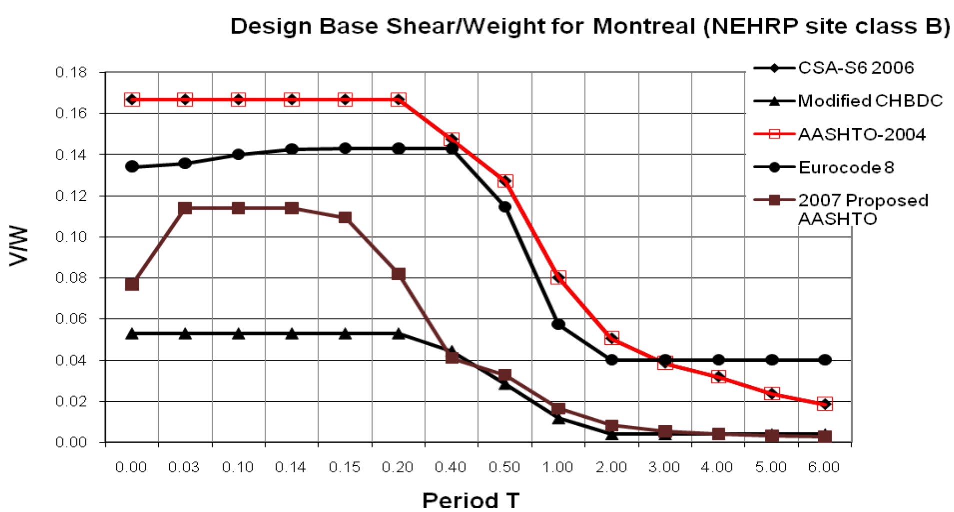



Figure 4.17 Design Base Shear/Weight for Montreal-NEHRP Site Class B

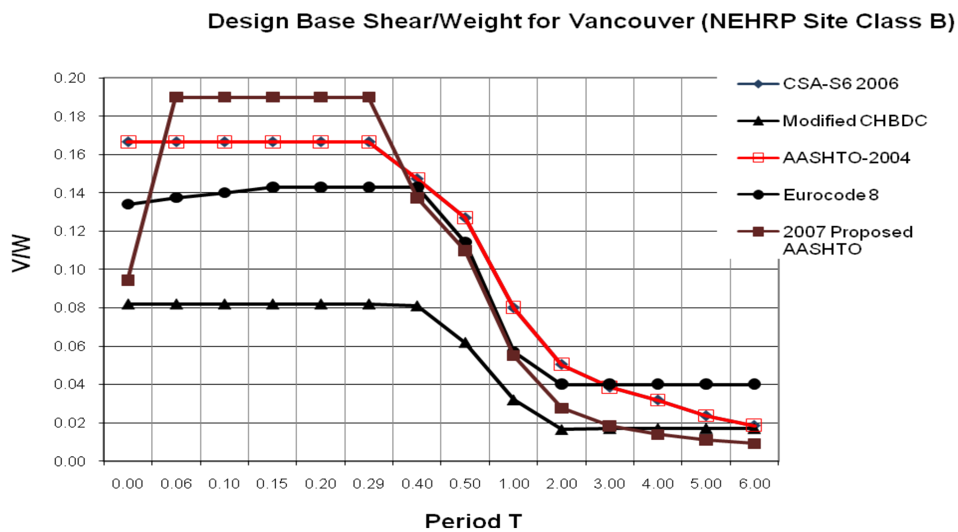



Figure 4.18 Design Base Shear/Weight for Vancouver-NEHRP Site Class B

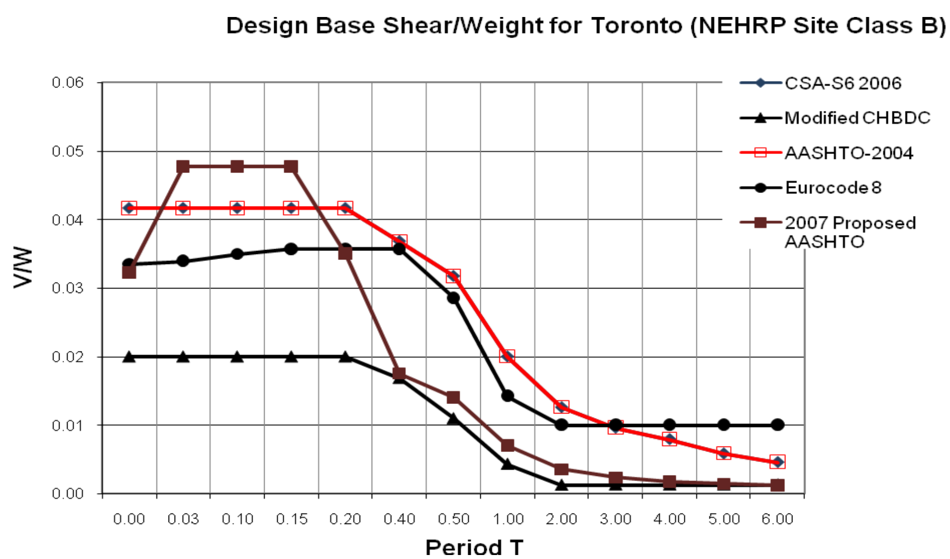



Figure 4.19 Design Base Shear/Weight for Toronto-NEHRP Site Class B

Figures 4.20, 4.21 and 4.22 contain the seismic design base shear per unit weight for Montreal, Vancouver and Toronto respectively, for NEHRP Soil Class E.

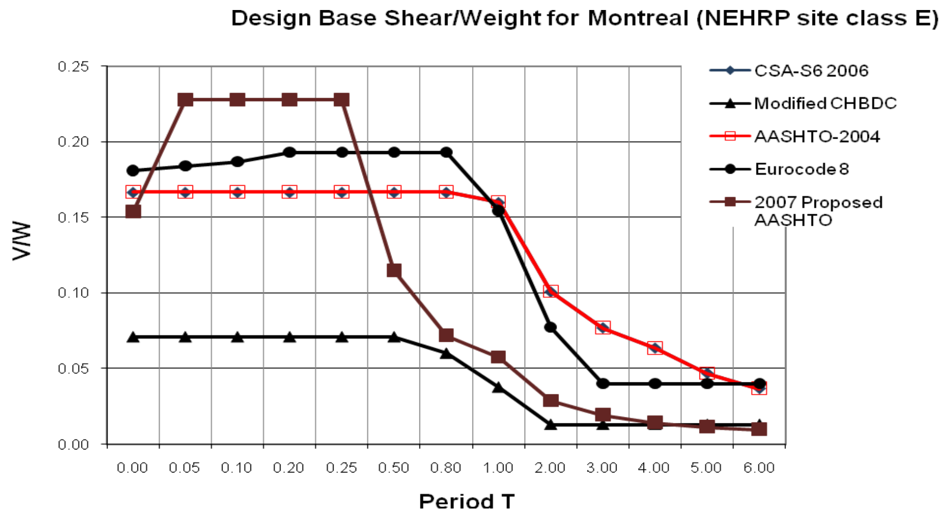



Figure 4.20 Design Base Shear/Weight for Montreal-NEHRP Site Class E

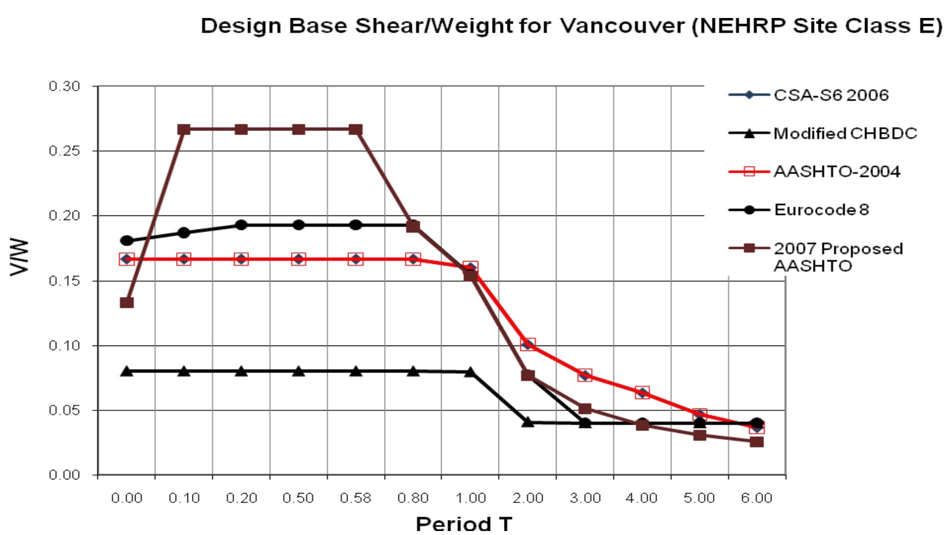



Figure 4.21 Design Base Shear/Weight for Vancouver-NEHRP Site Class E

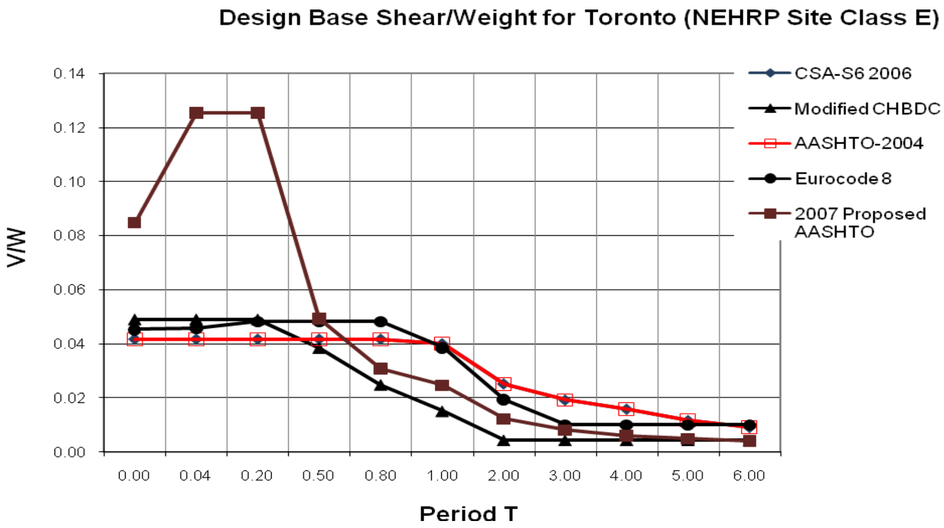



Figure 4.22 Design Base Shear/Weight for Toronto-NEHRP Site Class E

### 4.1.6 Seismic Displacement Check

The proposed AASHTO provisions include the following structure verification (Imbsen, 2007):

$$\Delta_D < \Delta_C \qquad [4.12]$$

Where:

$\Delta_D$ = displacement demand taken along the local principal axis of the ductile member

$\Delta_C$ = displacement capacity taken along the local principal axis corresponding to $\Delta_D$ of the ductile member

For Seismic Design Category (SDC) B:

$$\Delta_C = 0.12 H_o (-1.27 \ln(x) - 0.32) \geq 0.12 H_o \quad \text{(in)} \qquad [4.13]$$

For Seismic Design Category (SDC) C:

$$\Delta_C = 0.12 H_o (-2.32 \ln(x) - 1.22) \geq 0.12 H_o \quad \text{(in)} \qquad [4.14]$$

in which:

$$x = \frac{\Lambda B_o}{H_o} \qquad [4.15]$$

Where:

$H_o$ = clear height of column (ft.)

$B_o$ = column diameter (ft.)

Λ = factor for column end restraint condition

= 1 for fixed-free (pinned on one end)

= 2 for fixed top and bottom

Table 4.4 compares the seismic displacement demand against the calculated displacement capacity, for the proposed AASHTO values:

Table 4.4: Seismic Displacement Check

| City | Montreal | | Toronto | | Vancouver | |
|---|---|---|---|---|---|---|
| NEHRP Soil Class | B | E | B | E | B | E |
| Seismic Design Category | A | B | A | A | B | C |
| Displacement Demand (mm) | 27.2 | 96.2 | 11.6 | 43.3 | 93.9 | 256.5 |
| Displacement Capacity (mm) | 206.9 | 206.9 | 206.9 | 206.9 | 206.9 | 301.6 |

The Table above verifies the adequacy of the column section, when subjected to the various imposed displacements.

## 4.2 Discussion

### 4.2.1 Flexure and Shear

The results above highlight the differences between the seismic design provisions of the design codes investigated. The differences in the results are mainly attributed to the method used in defining the seismic hazard, and the overstrength factor used. For the bridge that was studied, the seismic design loads developed from the 2006 CHBDC and 2004 AASHTO are identical.

The design values of the CHBDC and AASHTO differ slightly from those of the Eurocode 8, primarily due to the effect of the overstrength and response modification factors. The aforementioned codes have similar design loads because they depend on ground accelerations for an event with a 475 year return period, to develop the seismic loads. The results developed from the CHBDC and AASHTO codes are used as a benchmark for comparing the other results.

The adoption of the spectral accelerations from the UHS and the overstrength factors decrease the seismic design loads significantly. The decrease is highlighted by the results for Montreal – NEHRP Soil Class B, where the modified CHBDC design moment is almost 8 % of the moment required by the CHBDC. The modified CHBDC results pertain to a seismic event with a 2500 year return period, which is more severe than the design event adopted by the 2006 CHBDC and 2004 AASHTO. Despite this factor, the spectral accelerations and the overstrength factor reduce the seismic design loads significantly, relative to those of the CHBDC. Accounting for the de-amplification effect of the subsurface characteristics (i.e., rock) on the seismic loads, and the shapes of the design spectra provide further explanations to the significant drop. It is believed that the design loads will be further reduced, had a smaller section been used to resist the lower modified CHBDC seismic loads. That is so because the smaller section will result in a more flexible structure with a longer natural period, and lower seismic design loads as a result. In most cases, the modified CHBDC values are less than those of the proposed AASHTO provisions, despite the lower earthquake return period adopted by the proposed provisions. The higher ductility factor and the use of the overstrength factor explain the above stated difference. The values of the modified CHBDC are significantly lower than the other spectra investigated, particularly beyond the one second period. This is exemplified with the graph for Toronto – NEHRP Soil Class E, where the spectrum is higher than that of the CHBDC/AASHTO, but decreases rapidly until it is lower than those spectra beyond the one second period.

The modified CHBDC and proposed AASHTO provisions have a more stringent requirement for bridges founded on Soil Class E. This is evident in the 270 % increase in seismic design loads for Toronto – NEHRP Soil Class E, relative to those for NEHRP Soil Class B. For the CHBDC, AASHTO and Eurocode 8, the difference between the seismic design loads of NEHRP Soil Classes B and E is less severe. The maximum increase between the seismic design loads of NEHRP Soil Class B and E for these codes is 100%.

Seismic design loads for the cities of Vancouver and Montreal are identical, according to the codes using only the ground acceleration as a seismic design parameter. For codes using the UHS, the seismic design loads for Vancouver are four times those of Montreal. This difference is attributed to the different ground motion characteristics in both cities. The design spectra for the city of Vancouver have a more gradual reduction as the period increases, compared to the reductions for the city of Montreal.

In all cases, the column transverse reinforcement was restricted by confinement requirements, and was not limited by the minimum shear demands. Another point of interest is the low seismic activity of Toronto. For regular type bridges in low seismicity regions, design codes generally adopt simplified design procedures to develop seismic loads (i.e., a percentage of the vertical load), in lieu of a rigorous seismic analysis procedure.

### 4.2.2 Ductility Demand

The displacement ductility for the single column used is 8.74, which highlights the degree of conservatism inherent in the response modification factor specified by the CHBDC, AASHTO and Eurocode 8. The ductility of the column used exceeds the ductility demand developed for all cases pertaining to NEHRP Soil Classes B and E. The ductility demands for NEHRP Soil Class E is almost double that of the NEHRP Soil Class B case, when the CHBDC and AASHTO code are employed. The ductility demand almost triples for all other investigated codes. There is no difference between the CHBDC, AASHTO and Eurocode 8 ductility demands for identical bridges found in Vancouver and Montreal. According to the modified CHBDC, the ductility demand in Vancouver is four times that of Montreal. This infers that the current CHBDC provisions necessitate that structures in Montreal be designed for four times its required ductility.

# Chapter 5 Conclusion

The following conclusions and recommendations result from the comparison of the seismic design provisions of the investigated design codes:

- Seismic design spectra based on the UHS spectral accelerations provide a more accurate and economically viable alternative to the current CHBDC design spectrum.
- The overstrength factor is an inherent design parameter that is included in seismic design codes in various ways. The overstrength factor has been included in the development of the loads and/or probable moments.
- The modified CHBDC seismic design provisions should replace the current seismic design approach with one involving the UHS spectral accelerations and overstrength factor. To adopt a uniform hazard approach, spectral accelerations based on a 2500 year return period should be employed in defining the seismic hazard.
- The effect of the soil conditions presented in the NBCC should be adopted by the CHBDC seismic subcommittee.
- Incorporate factors to account for different structural damping characteristics, dead load variation on the capacity of the column, vertical effect of earthquakes and varying ductility factors for structures with low periods.

# References

Adams, J., and Atkinson, G., 2003, “Development of Seismic Hazard Maps for the Proposed 2005 Edition of the National Building Code of Canada”, Canadian Journal of Civil Engineering, 30(2), pp. 255-271.

Adams, J., and Halchuk, S., 2003, “Fourth Generation Seismic Hazard Maps of Canada: Values for over 650 Canadian Localities intended for the 2005 National Building Code of Canada” Geological Survey of Canada Open File 4459. 155 p. Available from http://www.seismo/nrcan.gc.ca as of 1 April 2003.

American Association of State Highway and Transportation Officials, 2004, “AASHTO LRFD Bridge Design Specifications: U.S. Customary and SI units. (3rd ed.)”, Washington, D.C.: American Association of State Highway and Transportation Officials.

Blume, J. A., Newmark, N.M., and Corning, L. H., 1961, “Design of Multistory Reinforced Concrete Buildings for Earthquake Motions”, Chicago, I.L.: Portland Cement Association.

British Standards Institution and European Committee for Standardization, 2004, “Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings”, London: British Standards Institution.

British Standards Institution and European Committee for Standardization, 2005, “Eurocode 8 - Design of Structures for Earthquake Resistance – Part 2: Bridges”, London: British Standards Institution.

Building Seismic Safety Council, 2003, “NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450)”, Washington, D.C.: Building Seismic Safety Council.

Canadian Standards Association, 2006, “Commentary on CAN/CSA-S6-06, Canadian Highway Bridge Design Code (3rd ed.)”, Toronto: Canadian Standards Association.

Canadian Standards Association and Standards Council of Canada, 2006, “Canadian Highway Bridge Design Code”, Mississauga, Ont.: Canadian Standards Association.

Canadian Standards Association, 2004, “Design of Concrete Structures”, Mississauga, Ont.: Canadian Standards Association.

DeVall, R. H., 2003, “Background Information for Some of the Proposed Earthquake Design Provisions for the 2005 Edition of the National Building Code of Canada”, Canadian Journal of Civil Engineering, 30(2), pp. 279-286.

Fafitis, A., and Shah, S.P., 1985, “Lateral Reinforcement for High-Strength Concrete Columns”, ACI Spec. Publ., SP 87-12, pp. 213-232.

Fardis, M.N., Carvalho, E., Alnashai, A., Faccioli E., Pinto, P., Plumier, A., 2005, “Designers’ Guide to EN 1998-1 and EN 1998-5 : Eurocode 8 : Design of Structures for Earthquake Resistance : General Rules, Seismic Actions, Design Rules for Buildings, Foundations and Retaining Structures”, London: Thomas Telford.

Heidebrecht, A. C., 2003, “Overview of Seismic Provisions of the Proposed 2005 Edition of the National Building Code of Canada”, Canadian Journal of Civil Engineering, 30(2), pp. 241-254.

Humar, J., and Mahgoub, M. A., 2003, “Determination of Seismic Design Forces by Equivalent Static Load Method”, Canadian Journal of Civil Engineering, 30(2), pp. 287-307.

Imbsen & Associates, Inc., 2007, “Task 6 Report for Updating 'Recommended LRFD Guidelines for the Seismic Design of Highway Bridges’ ”, NCHRP 20-07/Task 193.

Imbsen, Roy A., 2007, “Proposed AASHTO Guide Specifications for LRFD Seismic Bridge Design”, Subcommittee for Seismic Effects on Bridges T-3.

Légeron, F. and Paultre, P., 2003, “Uniaxial Confinement Model for Normal and High-Strength Concrete Columns”, ASCE, Journal of Structural Engineering, 129(2), pp. 241-252.

Mander, J.B., Priestly, MJN, and Park, R., 1988, “Theoretical Stress-Strain Model for Confined Concrete”, American Society of Civil Engineers, 114 (8), pp. 1804-1826.

Mitchell, D., Bruneau, M., Buckle, I., Bagnariol, D., Zhu, S., and McCammon, N., 1998, “Developments in Short and Medium Span Bridge Engineering ‘98” Proceedings of the International Association of Bridge and Structural Engineering, International Conference on Developments in Short and Medium Span Bridges, Plenary Session.

Mitchell, D., Paultre, P., 1994, “Ductility and Overstrength in Seismic Design of Reinforced Concrete Structures” Canadian Journal of Civil Engineering, 21(6), pp. 1049-1060.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., and Anderson, D. L., 2003, “Seismic Force Modification Factors for the Proposed 2005 Edition of the National Building Code of Canada”, Canadian Journal of Civil Engineering, 30(2), pp. 308-327.

Morrison Hershfield Limited Consulting Engineers, 1991, “Ramp 427S-407W Overpass at Highways 427 & 407 - Highway 407/427 Interchange General Arrangement”.

Multidisciplinary Center for Earthquake Engineering Research (MCEER) and Applied Technology Council (ATC), 2003, “Design Examples-Recommended LRFD Guidelines for the Seismic Design of Highway Bridges”, Prepared under NCHRP Project 12-49, FY’ 98.

National Research Council of Canada, Canadian Commission on Building and Fire Codes and Institute for Research in Construction (Canada), 2005, “National Building Code of Canada (NBCC)”, Ottawa: National Research Council of Canada.

National Research Council of Canada, Canadian Commission on Building and Fire Codes and Institute for Research in Construction (Canada), 2006, “User's Guide - NBC 2005: Structural Commentaries (Part 4 of division B) (2nd ed.)”, Ottawa: National Research Council of Canada.

National Resources Canada, Interpolate 2005 National Building Code of Canada seismic hazard values for your site. 2008, from http://earthquakescanada.nrcan.gc.ca/hazard/interpolator/index_e.php.

Naumoski, N., Cheung, M. S., and Foo, S., 2000, “Evaluation of the Seismic Response Coefficient Introduced in the Canadian Highway Bridge Design Code”, Canadian Journal of Civil Engineering, 27(6), pp. 1183-1191.

NCHRP 12-49, 2001, “Comprehensive Specification for the Seismic Design of Bridges”, Third draft of specifications and commentary, Sponsored by AASHTO and FHWA.

Newmark, N. M. and Rosenblueth, E., 1971, “Fundamentals of Earthquake Engineering”, Englewood Cliffs, N.J.: Prentice-Hall.

Park, R., & Paulay, T., 1975, “Reinforced Concrete Structures”, New York: Wiley.

Paultre, P., Légeron, F. and Mitchell, D., 2007, “Design Confinement Reinforcement for Reinforced Concrete Columns”, Ninth Canadian Conference on Earthquake Engineering, Ottawa, Ontario.

Paultre, P, Mitchell, D., 2003, “Code Provisions for High-Strength Concrete: An International Perspective”, Concrete International vol. 25(No.5), pp. 76-90.

Popovics, S., 1973, “A Numerical Approach to the Complete Stress-Strain Curve of Concrete”, Cement and Concrete Research, 3(5), pp. 583-599.

Priestley, M. J. N., Seible, F., & Calvi, G. M., 1996, “Seismic Design and Retrofit of Bridges”, New York: Wiley.

Priestley, M. J. N., Calvi, G.M., Kowalsky, M.J., 2007, "Displacement-Based Seismic Design of Structures", Pavia, Italy: IUSS Press.

RESPONSE 2000 Version 1.0.5, Bentz, E.C.; Collins, M.P., University of Toronto, ON., Canada.

SAP2000 Advanced version 11, Computers and Structures, Inc., Berkeley, CA 94704.

# Appendix A Displacement Ductility of Confined Concrete Column

## A-1.1 Confined Concrete Stress-Strain Relationship

The following equations are used to develop the stress-strain relationship for confined concrete (Légeron and Paultre, 2003; Paultre et al., 2007):

The effective volumetric ratio of transverse reinforcement is determined as follows:

$$\rho_{se} = K_e \rho_s = (0.9)(0.0072) = 0.0065$$

Where:

$\rho_{se}$ = effective volumetric ratio of transverse reinforcement

$K_e$ = geometrical effectiveness coefficient, determined as the product of the vertical arching coefficient, $K_v$ = 0.9, and horizontal arching coefficient $K_h$ = 1.0

$\rho_s$ = volumetric ratio of the transverse reinforcement.

$$\rho = \frac{4A_s}{D's} = \frac{4*200mm^2}{2224mm*50mm} = 0.0072$$

To determine if the transverse reinforcement yields, the following equations are used:

$$\kappa = \frac{f'_c}{\rho_{se}E_s\varepsilon'_c} = \frac{35MPa}{0.0065*200000MPa*0.0035} = 7.7$$

$$f'_h = \frac{0.25 f'_c}{\rho_{se}(\kappa - 10)} \geq 0.43\,\varepsilon'_c\,E_s \leq f_{yield}\ \mathit{For}\ \kappa > 10\ \mathit{and}\ f'_h = f_{yield}\ \mathit{for}\ \kappa \leq 10$$

Since κ is less than 10, the effective stress in the transverse reinforcement is:

$$f'_h = f_{yield} = 400MPa$$

$$I'_e = \frac{f'_{le}}{f'_c} = \frac{\frac{1}{2}\rho_{se} f'_h}{f'_c} = \frac{\frac{1}{2} * 0.0065 * 400MPa}{35MPa} = 0.037$$

The maximum confined concrete stress and strain are determined as follows:

$$f'_{cc} = [1 + 2.4(I'_e)^{0.7}]f'_c = \left[1 + 2.4(0.037)^{0.7}\right]35MPa = 43.3MPa$$

The strain at that stress is:

$$\varepsilon'_{cc} = [1 + 35(I'_e)^{1.2}]\varepsilon'_c = \left[1 + 35\,(0.037)^{1.2}\right]0.0035 = 0.0058$$

The strain at 50% of the maximum confined concrete is determined as follows:

$$I_{e50} = \frac{1}{2}\rho_{se}\frac{f_{yield}}{f'_c} = 0.037$$

$$\varepsilon_{cc50} = (1 + 60 I_{e50})\varepsilon_{c50} = (1 + 60(0.037))0.004 = 0.0128$$

The ascending branch of the confined stress-strain curve is defined using the following equations (Popovics, 1973):

$$f_{cc} = f'_{cc}\left[\frac{k\left(\frac{\varepsilon_{cc}}{\varepsilon'_{cc}}\right)}{k - 1 + \left(\frac{\varepsilon_{cc}}{\varepsilon'_{cc}}\right)^k}\right] = 43.3\left[\frac{1.4\left(\frac{\varepsilon_{cc}}{0.0058}\right)}{0.4 + \left(\frac{\varepsilon_{cc}}{0.0058}\right)^{1.4}}\right]$$

$$k = \frac{E_{ct}}{E_{ct} - \left(\frac{f'_{cc}}{\varepsilon'_{cc}}\right)} = \frac{26{,}275MPa}{26{,}275MPa - \frac{43.3MPa}{0.0058}} = 1.4$$

The postpeak branch of the confined stress-strain curve is defined using the following equations (Fafitis and Shah, 1985; Légeron and Paultre, 2003):

$$f_c = f'_{cc} \exp\left[k_1 (\varepsilon_{cc} - \varepsilon'_{cc})^{k_2}\right] = 43.3 \exp\left[-115(\varepsilon_{cc} - 0.0058)^{1.03}\right]$$

$$k_1 = \frac{\ln 0.5}{(\varepsilon_{cc50} - \varepsilon'_{cc})^{k_2}} = \frac{-0.69}{(0.0128 - 0.0058)^{1.03}} = -115$$

$$k_2 = 1 + 25(I_{e50})^2 = 1 + 25\,(0.037)^2 = 1.03$$

### A-1.2 Displacement Ductility

The displacement ductility of the column investigated is developed as follows:

$$\mu_{D\,capacity} = \frac{\Delta_u}{\Delta_y}$$

$$\Delta_u = \Delta_y + \left[\left(\phi_u - \phi_y\right) L_p \left(H - \frac{L_p}{2}\right)\right]$$

Where:

$\mu_{D\ capacity}$ = column displacement ductility.

$\Delta_u$ = the ultimate displacement of the column.

$\Phi_u$ = the ultimate curvature of the column. Values of $\Phi_u$ were obtained from the Response2000 Moment – Curvature graph (Bentz and Collins, 2000), and are presented in Table 4.1.

$\Delta_y$ = the displacement at yield.

$\Phi_y$ = the curvature of the column at yield.

H = clear height of the column.

$L_p$ = length of plastic hinge in the column