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IMPACT OF CHANGES IN THE 2015 NBCC AND 2014 CSA A23.3 ON SEISMIC DESIGN FORCES FOR CONCRETE STRUCTURES

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ABSTRACT: A number of changes have been proposed for the 2015 National Building Code of Canada (NBCC) earthquake provisions. These include revisions to the seismic hazard values, new period-dependent site coefficients, revised higher mode and overturning moment factors and new short-period cut-offs. New provisions in the CSA A23.3-14 design standard also now explicitly quantify the required amplification of the design shear force to account for inelastic effects of higher modes. This paper presents an overview and discusses the impact of these proposed changes by comparing the 2015 NBCC and 2010 NBCC design base shear calculated using the equivalent static force procedure (ESFP) for various ductile reinforced concrete seismic force resisting systems (SFRSs). The comparison explores the full range of fundamental periods and considers various Site Classes in major urban centres, including Vancouver, Montreal, Ottawa, Victoria, Toronto and Québec. The effects of the inelastic higher mode factor on shear design, prescribed by the CSA A23.3-14 design standard, are also discussed.

1. Introduction

The Standing Committee on Earthquake Design (SCED) is responsible for developing earthquake design provisions in the National Building Code of Canada (NBCC). Major changes to the seismic design provisions have been proposed for inclusion in the 2015 NBCC. Some of these changes include revised seismic hazard values, new period-dependent site coefficients, revised design spectral acceleration values, revised higher mode and overturning moment factors, new hazard cap values for both the static and dynamic base shear procedures and new seismic requirements in low hazard zones. This paper provides an overview of some of the proposed changes affecting the determination of the seismic design shear forces using the equivalent static force procedure (ESFP) and presents a comparison of the 2010 and 2015 NBCC design base shears for two types of concrete seismic force resisting systems: ductile moment-resisting frames and ductile shear wall structures. This study considers various Site Classes over the full range of fundamental periods in Canada's most at risk urban centres: Vancouver, Montreal, Ottawa, Victoria, Toronto and Québec (Adams et al. 2002). The effects of inelastic higher modes on the design shear forces for shear walls, prescribed by the concrete design standard CSA A23.3-14, are also presented.

2. Proposed changes to the 2015 NBCC static earthquake load equations

The NBCC allows the use of the ESFP when certain conditions are met. These conditions in the 2015 NBCC remain unchanged from 2010. The following sections explore the changes to the 2015 NBCC static earthquake load equations and their parameters for determining the minimum base shear for design.

2.1. Minimum static lateral earthquake force

Three equations are included in the NBCC for the determination of the minimum static lateral earthquake force to be considered for design. As shown in Eq. 1, the form of the equation for the lateral earthquake force, V, in the 2015 NBCC remains unchanged from 2010.

$$V = \frac{S(T_a) M_v I_E W}{R_d R_o} \tag{1}$$

where

- $S(T_a)$ = design spectral acceleration, expressed as a ratio to gravitational acceleration, at the fundamental period, T_a, of the structure,
- $F(T)$ = period-dependent site coefficient,
- I_E = earthquake importance factor of the structure,
- M_v = factor to account for (elastic) higher mode effect on base shear,
- W = dead load plus 25% of the design snow load plus 60% of the storage load,
- R_d = ductility-related force modification factor, and
- R_o = overstrength-related force modification factor.

As in 2010, the 2015 NBCC also prescribes lower and upper bounds for the lateral earthquake force, V. The procedure for determining the minimum lateral earthquake force remains unchanged from 2010. For walls, coupled walls and wall-frame systems, V shall not be less than that calculated from Eq. 1 using $S(T_a)$ equal to S(4.0). For moment-resisting frames, V shall not be less than that calculated from Eq. 1 using $S(T_a)$ equal to S(2.0).

In the 2010 NBCC, the design base shear for buildings located on a site other than Class F with an R_d equal to or greater than 1.5 did not have to be taken greater than:

$$V = \frac{2}{3} \frac{S(0.2) I_E W}{R_d R_o} \tag{2}$$

As the new 2015 NBCC spectral values often resulted in flatter spectrums, the 2/3S(0.2) cut-off limit shown in Eq. 2 extended in many cases beyond periods of 0.5s where a reduction in design base shear is not warranted. To ensure that the short period cut-off remained for the intended short period structures, a second cut-off condition, S(0.5), as shown in Eq. 3 was included in the 2015 NBCC to cover cases where the 2/3S(0.2) limit extended beyond the originally intended 0.5s period. Therefore, the proposed change in the 2015 NBCC now requires that V need not be taken greater than the larger of Equations 2 and 3.

$$V = \frac{S(0.5) I_E W}{R_d R_o} \tag{3}$$

The results of Equations 2 and 3 provide the new short-period cap in the 2015 NBCC for determination of the design base shear at low periods.

2.2. Revised seismic hazard values and new foundation factors

The Geological Survey of Canada has updated the seismic hazard values for the 2015 NBCC based on a probabilistic model using mean values (NRCC 2015). Values in the 2010 NBCC were specified for periods of 0.2, 0.5, 1.0 and 2.0s. The values in the 2015 now extend to include values for 5.0 and 10.0s. In addition, new period-dependent site coefficients, F(T), were developed for these corresponding periods for different Site Classes as a function of the reference peak ground acceleration, PGA_{ref}.

Despite the change in foundation factor methodology, both the 2010 acceleration-based and velocity-based site coefficients, F_a and F_v, were retained in the 2015 NBCC. These factors are still used for the purpose of calculating the short and long period spectral design acceleration “triggers”, $I_E F_a S_a(0.2)$ and $I_E F_v S_a(1.0)$, respectively, for determining applicable SFRS and design restrictions.

2.3. Changes to the calculation of the design spectral values

As a result of the updated $S_a(T)$ values and the new $F(T)$ values, the design spectral response acceleration, $S(T)$, in the 2015 NBCC is determined by the product of $F(T)$ and $S_a(T)$. The values of $S(T)$ for different period ranges are given as:

$$
\begin{aligned}
S(T) = \;& F(0.2)S_a(0.2) \text{ or } F(0.5)S_a(0.5) \text{ whichever is larger for } T \le 0.2\text{ s} \\
& F(0.5)S_a(0.5) \text{ for } T = 0.5\text{s} \\
& F(1.0)S_a(1.0) \text{ for } T = 1.0\text{s} \\
& F(2.0)S_a(2.0) \text{ for } T = 2.0\text{s} \\
& F(5.0)S_a(5.0) \text{ for } T = 5.0\text{s} \\
& F(10.0)S_a(10.0) \text{ for } T \ge 10.0\text{s}
\end{aligned} \tag{4}
$$

As noted in Equation 4, the design spectral acceleration for a period of 0.2s is to be taken as the larger of that calculated at 0.2s and 0.5s. For some locations S(0.5) is greater than S(0.2) and it is not prudent to design on the basis of a spectrum in which the S value increases with increasing period. This is due to the fact that the fundamental period of a structure could increase as the SFRS softens under cyclic loading and hence could attract higher forces.

It is also noted that the S(T) expressions in Equation 4 have been extended to 10.0s in the 2015 NBCC in light of the new $S_a(T)$ values.

Table 1 – Proposed new seismic force resisting systems in the 2015 NBCC for concrete structures designed and detailed according to CSA A23.3-14 (Adapted from NRCC 2015).

Type of SFRS	R_d	R_o	Restrictions				
			Cases where $I_E F_a S_a(0.2)$				Cases where $I_E F_v S_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Moderately ductile coupled walls	2.5	1.4	NL	NL	NL	60	60
Moderately ductile partially coupled walls	2.0	1.4	NL	NL	NL	60	60
Conventional construction							
Two-way slabs without beams	1.3	1.3	20	15	NP	NP	NP
Tilt-up construction							
Moderately ductile walls and frames	2.0	1.3	30	25	25	25	25
Limited ductility walls and frames	1.5	1.3	30	25	20	20	20[1]
Conventional walls and frames	1.3	1.3	25	20	NP	NP	NP

(1) maximum of 3 storeys

2.4. Revised higher mode and overturning moment factors

Both the higher mode factor, M_v, and the base overturning moment reduction factor, J, have been revised in the 2015 NBCC to account for new spectral shapes, soil Site Classes and have been extended to structural periods of 5.0s and greater. The factors are now expressed in terms of the spectral ratio S(0.2)/S(5.0), varying from 5 to 65. It is noted that the NBCC M_v factor accounts for the higher mode effects determined from dynamic analysis considering elastic response.

2.5. New concrete seismic force resisting systems

Six new concrete seismic force resisting systems (SFRSs) have been proposed for inclusion in Table 4.1.8.9 of the 2015 NBCC. These include moderately ductile coupled and partially coupled shear walls, conventional two-way slab construction without beams and three categories of tilt-up wall and frame systems. These new SFRSs with corresponding R_d and R_o values along with restrictions and height limits in metres are shown in Table 1. Corresponding design and detailing requirements for these new systems were developed and introduced into CSA A23.3-14 Standard.

3. Comparison of base shears from the 2010 and 2015 NBCC

In order to assess the impact of changes in the 2015 NBCC on the design of reinforced concrete structures, NBCC base shear values V/W were computed based on the ESFP and compared. This comparison was carried out for an earthquake importance factor, I_E, of 1.0.

Figures 1 and 2 compare the 2010 and 2015 NBCC values of V/W for concrete ductile shear wall and ductile moment-resisting frame SFRSs, respectively. The values of V/W are given for Site Classes A, C and E, and for different cities. The locations chosen represent the Canadian urban centres considered to be “most at risk” according to Adams et al. (2002). The 2015 NBCC values of V/W are influenced by the changes to the seismic hazard, the period-dependent foundation factors, the higher mode factor, the short-period cap and the type of SFRS.

3.1. Ductile shear walls

As shown in Figs. 1(a) and (b), the values of V/W for ductile shear wall structures for Victoria and Vancouver have generally increased in the 2015 NBCC. For example, at a period of 2.0s, the V/W values for Victoria have increased by 50, 92 and 65% for Site Classes A, C and E, respectively. The corresponding increases for Vancouver are 12, 32 and 23%.

Figure 1(c) illustrates that V/W values for Toronto have not changed as significantly as in the other cities shown in Fig. (1). The largest increase of 17% occurs at a period of about 1.0s for Site Class E. As shown in Figure 1(d), the V/W ratios for Ottawa have overall decreases ranging from 3% at a period of 1.0s for Site Class A up to 36% at a period of 4.0s for Site Class E.

The V/W ratios for Montreal have decreased at periods equal to and less than 0.5s (see Fig. 1(e)). These ratios have increased at periods greater than about 0.5s for Site Classes A and C. The values for Site Class E have decreased by 10 to 29%. As shown in Figure 1(f), the V/W ratios for Quebec City have overall decreases ranging from 1% at a period of 1.0s for Site Class A up to 38% at a period of 4.0s for Site Class E.

3.2. Ductile moment-resisting frames

It is noted that the higher mode factor, M_v, is equal to 1.0 for both walls and moment-resisting frames for periods equal to or less than 0.5s. Hence, the V/W ratios for these two SFRSs are the same for a given location and Site Class.

The values of V/W for Victoria have increased by 6% at a period of 0.2s for Site Class A up to 116% at periods of 2.0s and greater for Site Class C (see Fig. 2(a)). For Vancouver, the V/W values for Site Class A have increased by 12% for periods of 2.0s and greater and have decreased by 14% for a period of 0.2s (see Fig. 2(b)). For Site Classes C and E, V/W ratios for periods of 2.0s and greater have increased by 45 to 49%. For Site Class E, V/W values at periods of 0.5s or less have increased by 72%.

Figure 2(c) illustrates that changes in V/W values for Toronto range from a decrease of 23% at a period of 1.0s for Site Class A to an increase of 49% at periods of 2.0s or greater for Site Class E. As shown in Figure 2(d), the V/W ratios for Ottawa have decreased by 15 to 34% except for periods of about 2.0s or greater for Site Classes C and E where values have slightly increased by 1 to 6%.

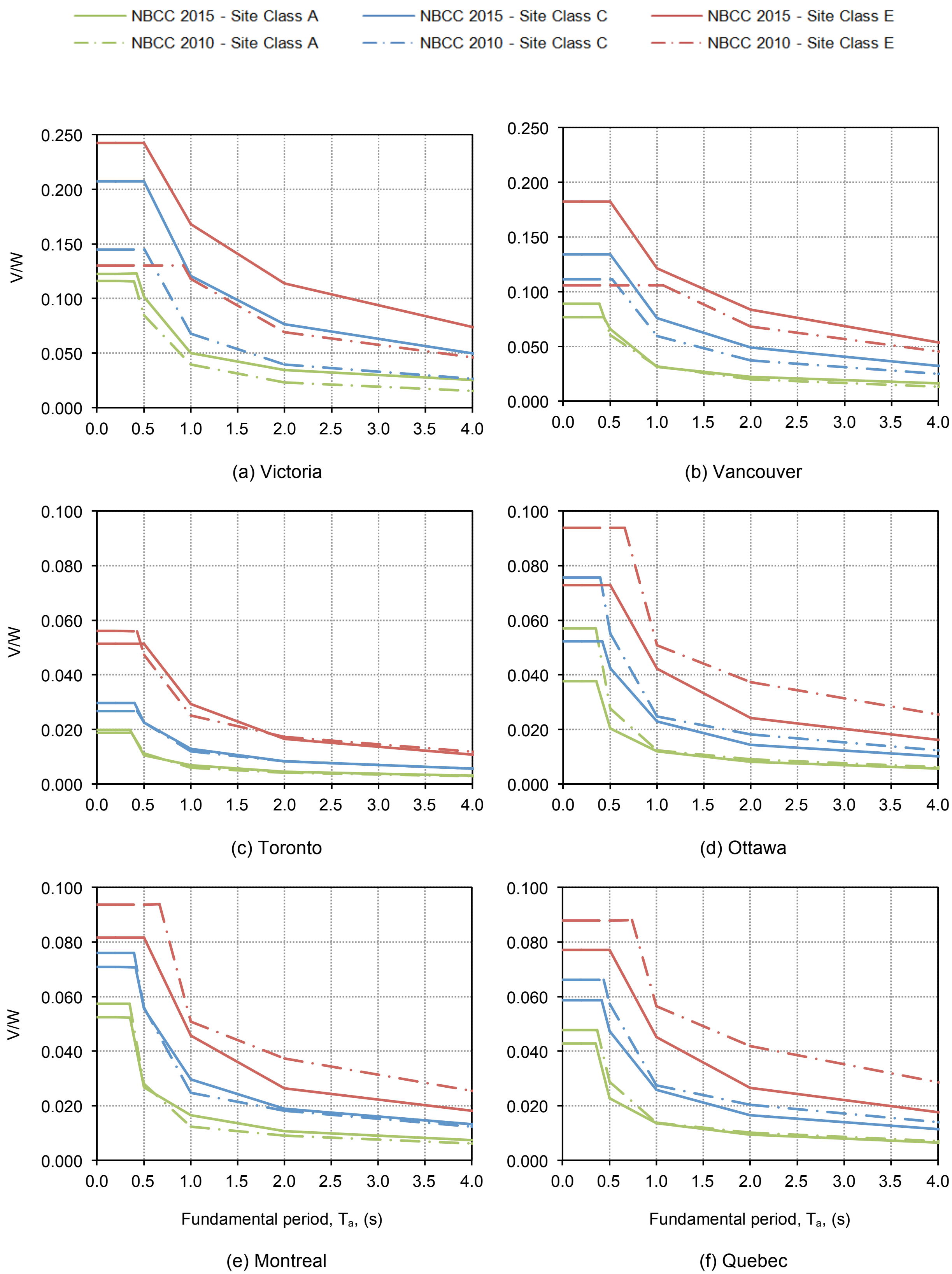

Fig. 1 – Comparison of 2010 NBCC and 2015 NBCC V/W Ratios for Concrete Ductile Shear wall SFRS For Site Class A, C and E

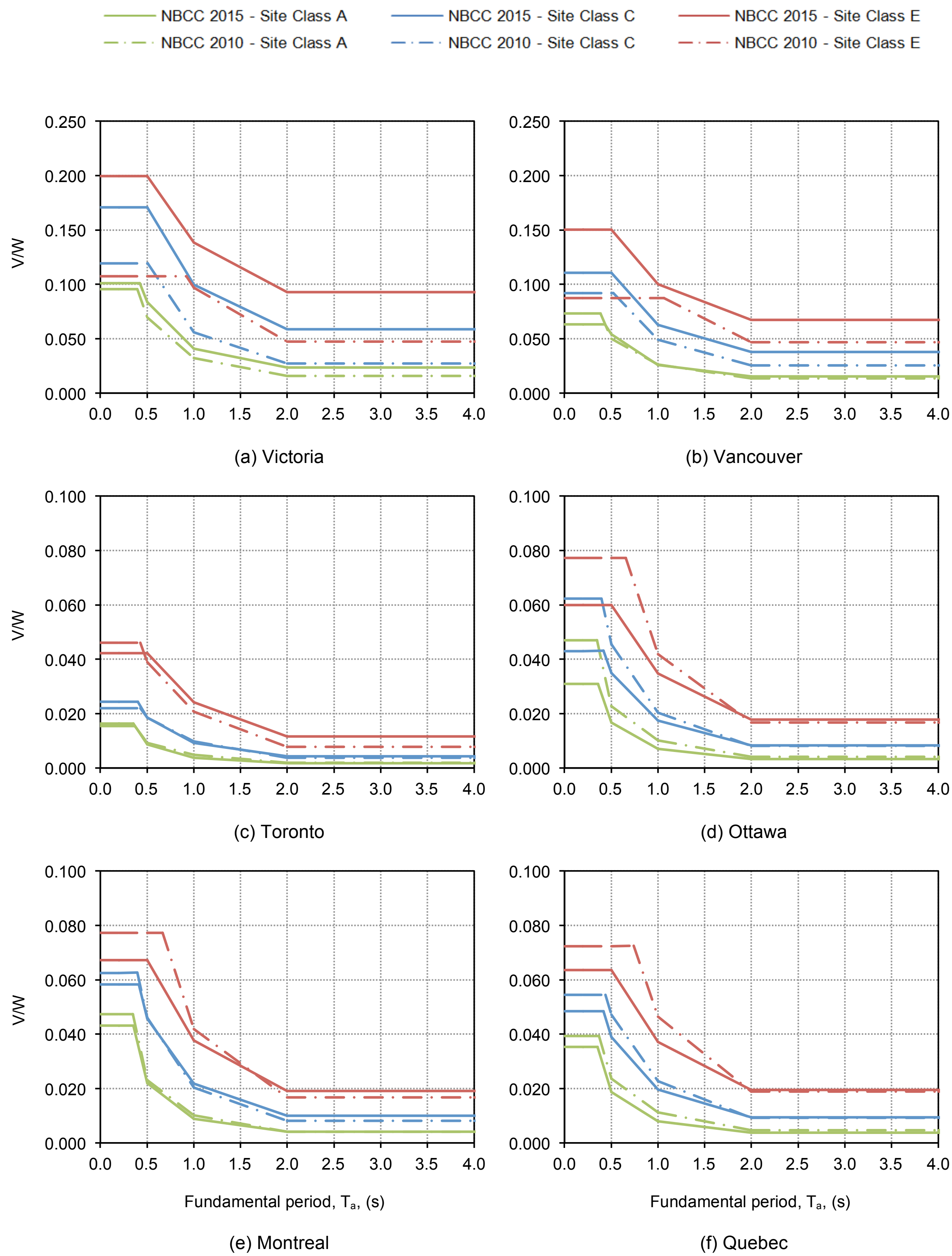

Fig. 2 – Comparison of 2010 NBCC and 2015 NBCC V/W Ratios for Concrete Ductile Moment-Resisting Frame SFRS For Site Class A, C and E

The V/W ratios for Montreal have decreased by 1 to 12% for Site Class A for periods greater or equal to 1.0s (see Fig. 2(e)). For periods of about 2.0s or greater for Site Classes C and E, values have increased by 14 to 23%. As shown in Figure 2(f), the V/W ratios for Quebec City have decreased by 10 to 29% except for periods of about 2.0s or greater for Site Classes C and E where values have increased slightly by 3 to 4%.

4. New provision in CSA A23.3-14 to account for inelastic effects of higher modes for shear walls

Although the 2004 CSA A23.3 provisions required that the designer "shall account for the magnification of the shear due to the inelastic effects of higher modes" for shear walls, no guidance was provided. Provisions in the CSA A23.3-14 concrete design standard now explicitly quantify the required amplification of the design shear force to account for these effects (CSA 2014). The required amplification is a function of the ductility- and overstrength-related force modification factors (R_d and R_o) for the SFRS, the wall overstrength factor (γ_w), the spectral shape ratio (S(0.2)/S(2.0)) and the fundamental period (T_a) of the structure. The amplification is determined from Tables 2 and 3.

Table 2 – Inelastic higher mode factor, ω (CSA 2014).

$T_a \leq T_L$	$T_a \geq T_U$
1.0	$1.0 + 0.25\left(\frac{R_d R_o}{\gamma_w} - 1\right) \leq 1.5$ and ≥ 1.0

Table 3 – Inelastic higher mode period parameters (CSA 2014).

	T_L	T_U
S(0.2)/S(2.0)<10.0	0.5 s	1.0 s
S(0.2)/S(2.0)≥10.0	0.2 s	0.5 s

It is noted that the provisions pertaining to the inelastic higher mode effects applies for shear wall SFRSs with a ductility, R_d, of 2.0 or greater. The amplification factor, ω, does not apply to coupled or partially coupled shear walls. Figure 3 shows the variation of the inelastic higher mode amplification factor, ω, as a function of the fundamental period of the structure, T_a, and the product R_dR_o. The solid lines are for cases with spectral acceleration ratios, S(0.2)/S(2.0), less than 10.0 whereas the dashed lines are for spectral ratios greater than or equal to 10.0. As illustrated in the figure, ω varies from 1.0 to 1.29 for moderately ductile shear walls and from 1.0 to 1.5 for ductile shear walls. It is also important to note that the inelastic higher mode shear amplification is in addition to the higher shear resulting from the flexural overstrength magnification.

It is noted that the New Zealand Standard (NZS 2006) also requires that the design shear for walls be magnified by an overstrength factor and by a nonlinear dynamic amplification factor. The nonlinear dynamic amplification factor in the 2006 NZS is a function of the number of storeys.

5. Effects of changes to the NBCC and CSA A23.3 Standard on design shear forces for shear wall structures

There have been significant changes to the seismic design provisions in both the NBCC and the CSA A23.3 Standard. Mitchell et al. (2010) provided a comparison of seismic design base shears (V/W) for concrete wall and moment-resisting frame structures located in Vancouver and Montreal. Covering the period from 1941 to 2005, this comparison illustrates the evolution of the introduction of ductility and the corresponding design and detailing provisions of CSA A23.3 Standard. An overview of the seismic changes to the 2014 edition of CSA A23.3 is provided by Adebar et al. (2014).

In order to capture the impact of changes in the CSA A23.3-14 Standard, the study of the 2015 NBCC shear V/W values presented earlier was extended to account for the requirements of the Standard. Figure 4 compares the overall design base shear ratios (V/W) for concrete ductile shear wall structures calculated in accordance with the design provisions of the NBCC and CSA A23.3 standard used in 2010 and 2015 for Vancouver and Montreal. Concrete structures designed in accordance with the 2010 NBCC had to satisfy the design and detailing provisions of the CSA A23.3-04 Standard; whereas, those designed in accordance with the 2015 NBCC must satisfy the provisions of the CSA A23.3-14 Standard. The figure illustrates the combined effects of changes to seismicity, foundation factors, elastic higher mode factor, flexural overstrength and inelastic higher mode effects on the design shear force for ductile shear walls. The earthquake importance factor, I_E, was taken as 1.0 and a flexural overstrength factor of 1.47 (Mitchell and Paultre, 2006) was used for the comparison.

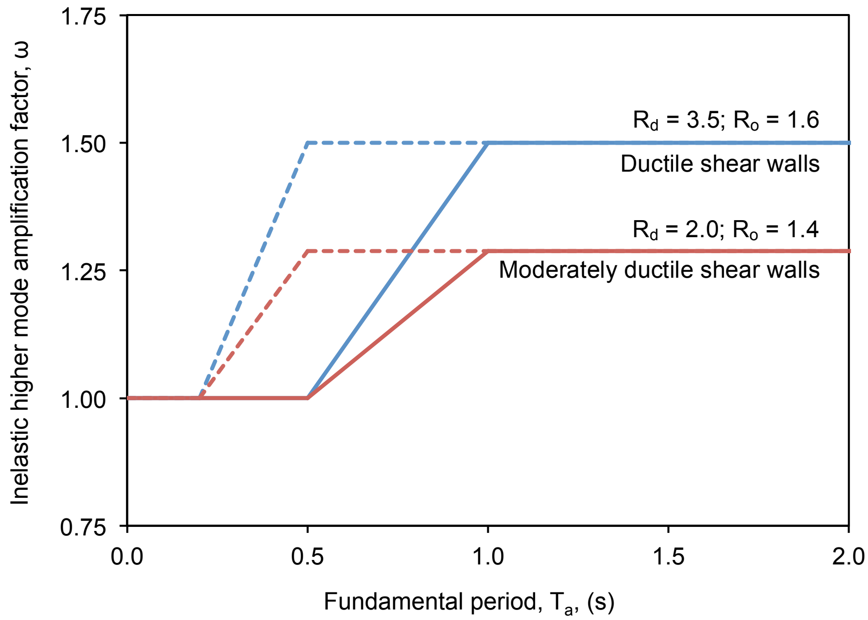

Fig. 3 – Inelastic higher mode shear amplification factor, *ω*

A stacked column format was used to highlight the various contributing factors to the overall design base shear values (V/W). The blue shaded columns represent V/W values calculated in accordance with the 2010 NBCC and/or CSA A23.3-04 requirements whereas the red shaded columns represent those values calculated in accordance with the 2015 NBCC and/or CSA A23.3-14 requirements. For the CSA A23.3-04 requirements, it was assumed that the effects of higher modes were not considered as no guidance was provided. The lightest shade in the stack represents the base shear force calculated in accordance with the NBCC provisions. It accounts for the effects of seismicity, site conditions, importance of the structure and the effects of elastic higher modes. The darker shade in the stack represents the amplification of the NBCC force for consideration of flexural overstrength as required by CSA A23.3. The darkest shade represents the newly quantified inelastic effects of the higher modes' provision in CSA A23.3-14.

Figures 4(a) and (b) illustrate the overall design shear V/W values when considering the combined NBCC and CSA A23.3 requirements for Vancouver for Site Classes A and E, respectively. As the spectral ratio S(0.2)/S(2.0) is less than 10 for all Site Classes, the inelastic higher mode amplification factor, ω, for periods equal to or less than 0.5s is 1.0. As such, the changes in V/W values at those periods are the same as those considering the NBCC requirements only. For Site Class A, the overall design shear (V/W) increased by 49, 68, and 84% for periods of 1.0s, 2.0s and 4.0s, respectively. Corresponding increases for Site Class E are 72, 84 and 78%.

The overall design shear V/W values for Montreal are shown in Figures 4(c) and (d) for Site Classes A and E, respectively. Despite a 5% decrease in the NBCC V/W base shear at 0.5s (see Fig. 1(e)) for Site Class A, the overall design shear V/W increased by 43% when considering the CSA A23.3-14 inelastic higher mode effect factor (see Fig. 4(c)). For periods of 1.0s, the overall design shear V/W values have increased by 100 and 35% for Site Classes A and E, respectively. The corresponding increases at periods of 2.0s and greater are about 78 and 7%.

It is important to note that the changes in this comparison are based on an assumed flexural overstrength factor. Greater changes in overall design shear forces may be expected for larger wall overstrength factors, that is, if the factored flexural resistance of the shear wall significantly exceeds the flexural resistance required to match the factored load.

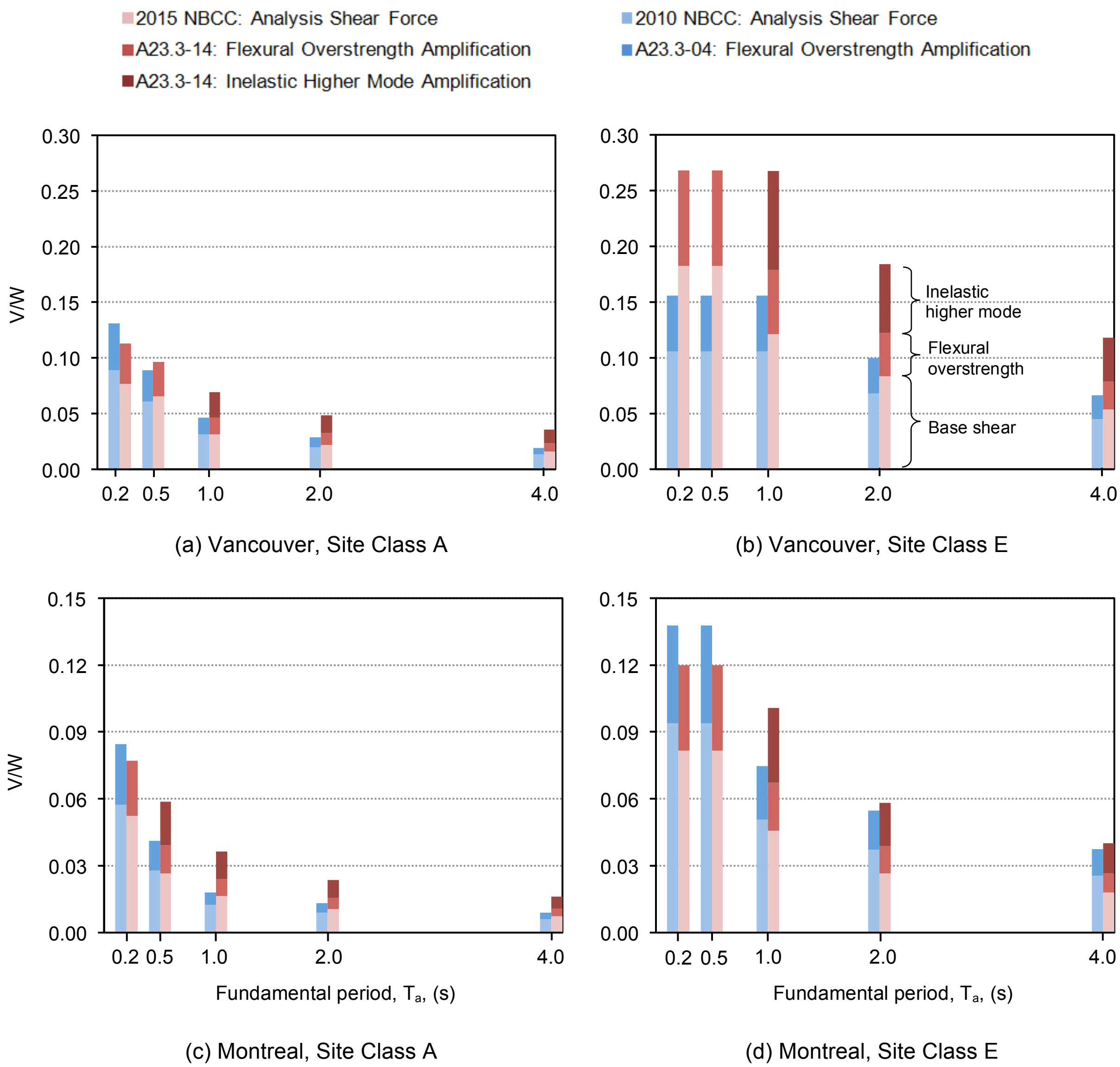

Fig. 4 – Comparison of Design V/W Ratios for Concrete Ductile Shear Walls For Site Classes A and E

6. Conclusions

Significant changes to the minimum lateral earthquake force equation parameters have been proposed for the 2015 NBCC. As shown, these changes can appreciably impact the NBCC design base shear for concrete structures. Out of the 6 locations studied, Victoria had the greatest impact. In addition, the comparison has shown that the quantification of the inelastic higher mode shear amplification factor in CSA A23.3-14 will significantly increase overall design shear forces for longer period concrete shear wall structures, with increases of roughly 30 and 50% for R_d values of 2.0 and 3.5, respectively.

7. Acknowledgements

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