Inelastic seismic shear amplification due to higher mode effects in reinforced concrete coupled walls

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Abstract
Recent numerical and experimental studies on reinforced concrete shear walls and coupled walls have shown shear forces greater than expected when the walls are subjected to earthquakes at an intensity level that does not exceed the design values. This amplification of shear forces is attributable to the effects of higher modes after the walls develop a plastic hinge at the base. These effects have been recently recognized in North American design codes for cantilever walls and is currently neglected in the design of ductile coupled walls. As part of the research program described in this article, a parametric study was carried out on coupled wall systems to identify the geometric and physical parameters having the greatest influence on the seismic shear amplification. Using the results of this parametric study, an extensive numerical study was conducted on classes of ductile coupled walls subjected to seismic excitation representative of Western and Eastern Canada. This extensive study led to the establishment of shear amplification prediction equations for use in building codes.

Keywords
Shear walls, coupled walls, reinforced concrete, capacity design, higher mode effects, non linear time history analysis, shear amplification

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Introduction
Capacity design of shear walls as the primary seismic force-resisting system (SFRS) is based on favoring a ductile mode of failure by flexure at the base of the walls while ensuring that other fragile modes of failure, such as by shear, are enforced to remain in the elastic range. This design is achieved by reducing the flexural demand by a force reduction

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factor, thus favoring the formation of a plastic hinge at the base of the shear walls while providing ductile flexural behavior through adequate reinforcement detailing in the so-called plastic hinge zone. The design shear envelope over the height of the walls is then based on the probable flexural capacity of the walls at their base, accounting for flexural overstrength. Although theoretically sound, inelastic seismic analyses of shear walls have shown that the shear force continues to increase long after the formation of a plastic hinge at the base. This behavior has been attributed to the effects of higher modes when yielding at the base of walls tends to elongate the first mode period, thus limiting the seismic shear demand of that mode, while the demand of the higher modes is increasing. An inelastic shear amplification due to higher modes effects has been recommended for a long time in New Zealand for cantilever walls and is now incorporated in the most recent American Concrete Institute (ACI) code. It was not before 2014 that an amplification factor, based on analytical research, was formally proposed for the determination of the design shear envelope for cantilever shear walls in the Canadian Standard. This recommendation does not apply to walls linked by stiff coupling beams, known as coupled walls. The main objective of the current research is to investigate if there is an inelastic higher mode shear amplification in coupled shear wall structures and, if so, to develop a predicting equation that can be adopted in design codes. To achieve this goal, a numerical parametric study was conducted to identify the main geometric and physical parameters controlling the inelastic higher mode shear amplification. Using the results of this first step, an extensive numerical study was conducted on a large range of coupled wall systems designed according to the Canadian codes to determine the shear amplification factors as a function of these controlling parameters.

**Capacity design of walls according to CSA A23.3-14 and NBCC-15**

Since the 2005 edition of the National Building Code of Canada (NBCC), the linear dynamic method has been recommended for the calculation of seismic forces. A combination of spectral responses is generally obtained based on the uniform hazard design spectrum (UHS), which represents the 5% damping response of a single degree of freedom oscillator to a suite of acceleration excitations having the same 2% probability of exceedance in 50 years. The NBCC-15 allows a reduction in seismic design forces to account for the ductility and overstrength of the structures:

\[ V = \frac{V_e}{R_d R_o} \]

where \( V \) is the factored seismic shear force, \( V_e \) is the elastic base shear obtained by the modal response spectrum analysis, \( R_d \) is the seismic force reduction factor related to the ductility, which varies between 1 and 4, and \( R_o \) is the overstrength-related reduction factor, which varies between 1 and 1.7. Note that the product \( R_d R_o \) corresponds approximately to the force reduction factor \( R \), and \( R_o \) to \( \Omega_o \), in the American Society of Civil Engineers (ASCE) 7 code (ASCE, 2016). Typical values of \( R_d \) and \( R_o \) for shear wall structures according to the NBCC 2015 are given in Table 1.

Capacity design is used in the Canadian and New Zealand codes to prevent undesirable failure mechanisms and allow energy dissipation through inelastic behavior. It is basically a three-step process (Maffei and Yuen, 2007; Paulay and Priestley, 1992):
1. Select a desirable ductile inelastic mode of deformation, which, in the case of a shear wall structure, is based on the formation of a flexural plastic hinge at the base of the wall;

2. Detail the walls to have sufficient ductility according to the ductility level $R_d$ selected in determining the seismic force demand;

3. Design all other elements and actions of the structures to remain in the elastic range. In the particular case of walls, shear behavior should remain elastic.

As mentioned, among the failure mechanisms, shear failure prior to the development of a plastic hinge at the base should be avoided since such failures occur suddenly and limit the ability of the structure to dissipate energy during subsequent loading cycles. To prevent these failure modes, the shear strength must be greater than the required demand to allow the development of the wall probable flexural capacity at the base. To ensure a hierarchy of failure of a structure, the A23.3-14 Design of Concrete Structure Standard (CSA, 2014) defines three classes of resistance: (1) the factored resistance, $R_f$, is calculated by applying partial safety factors $\phi_c = 0.65$ and $\phi_s = 0.85$ to the concrete and to the steel resistances, respectively; (2) the nominal resistance, $R_n$, is determined by considering that the materials have the specified ideal resistance using $\phi_c = \phi_s = 1.00$; and (3) the probable resistance, $R_p$, which considers strain hardening of the reinforcement and the fact that the actual yield strength is greater than the specified value. Hence, the flexural probable resistance of a shear wall, $M_p$, can be calculated approximately by applying an overstrength factor of 1.25 to the steel reinforcement yield strength, assuming ductile flexural behavior and using the specified concrete resistance $f'_c$. Since the walls are designed to be ductile and the amount of longitudinal reinforcement is usually small, less than 1%, their flexural behavior is controlled mainly by the reinforcement yield strength and the ratio of the probable to nominal resistance will be approximately equal to $1.25/1.0 = 1.25$, while the ratio of the probable to factored resistance will be approximately equal to $1.25/0.85 = 1.47$.

To prevent shear failure, capacity design requires the cantilever wall factored shear resistance to be sufficient to resist the shear force corresponding to the development of the probable flexural resistance at the base of the wall. Assuming a linear relationship between the shear force and the bending moment, this requirement is satisfied using the factored shear force envelope corresponding to the probable moment resistance at the base. The shear envelope is further increased by a factor of $\omega_v$ to account for the inelastic effects of higher modes. Hence, the amplified probable shear force demand, $V_{dp}^a$, at the base of a cantilever wall is calculated as:

$$ V_{dp}^a = \omega_v V_{dp} = \omega_v \left( \frac{M_p}{M_f} \right) V_f $$

### Table 1. Seismic force reduction factors $R_d$ and $R_o$ for some reinforced concrete structures in the NBCC-15 (National Research Council of Canada, 2015)

| SFRS                     | $R_d$ | $R_o$ |
|--------------------------|-------|-------|
| Ductile coupled walls    | 4.0   | 1.7   |
| Ductile partially coupled walls | 3.5   | 1.7   |
| Ductile shear wall       | 3.5   | 1.6   |
| Conventional construction shear walls | 1.5   | 1.3   |

NBCC: National Building Code of Canada; SFRS: seismic force-resisting system.
where \( V_{dp} = (M_p/M_f)V_f \) is the probable design shear envelope determined according to the capacity design method by amplifying the factored shear envelope \( V_f \) by the ratio of the probable moment resistance \( M_p \) over the design factored moment \( M_f \), both evaluated at the base of the wall, and \( \omega_v \approx 1.0 \) is the shear amplification factor due to the inelastic higher mode effects. Prior to the 2014 edition of the CSA A23.3 standard, no inelastic shear amplification factor was specified. Although the standard now proposes an expression for \( \omega_v \) for isolated structural walls, no shear amplification factor is recommended for coupled walls.

**Review of the inelastic higher mode response of reinforced concrete shear walls**

The linear dynamic behavior of structures can be described by the superposition of the responses of the different modes of vibration. These modes depend essentially on the mass and stiffness distribution over the entire height of the building. The equivalent static procedure proposed in building codes is based on the first mode response. This procedure is adequate for predicting the lateral displacement of multistory structures but not for the shear force distribution over the height of the structure, which depends on the frequencies squared and, therefore, more heavily on the higher modes. In the equivalent static force procedure, this distribution is usually addressed using an additional force at the top of the building and, sometimes, with an elastic shear amplification factor. Linear dynamic analysis using a combination of spectral responses is now the recommended procedure for seismic design in most modern codes, such as the 2015 National Building Code of Canada (NBCC-15; National Research Council of Canada, 2015), and these shear amplifications are automatically considered by using an appropriate number of modes whose sum of modal masses represents a specified minimum value of the total mass of the structure, usually 90%. This well-known phenomenon is not the subject of this research program, which concentrates on the unexpected additional shear amplification that comes after yielding at the base of shear walls designed according to the capacity design method, as shown by recent research. Recall that, in the capacity design method, the shear force is amplified to avoid shear failure after forming a flexural plastic hinge at the base of cantilever walls. At a given instant during seismic responses, when the flexural plastic hinge is fully developed, the shear force will be influenced much more by the higher modes, whose shapes and associated periods do not change considerably depending on whether the base of the wall is hinged or fixed, than by the first mode, whose shape changes and associated period lengthens when a plastic hinge is formed at the base, substantially reducing its participation. Another way to consider this problem is illustrated in Figure 1, which shows the distribution of seismic forces predicted by nonlinear dynamic analysis of a 12-story shear wall structure under the action of a given earthquake at the time of maximum shear force at the base (Boivin and Paultre, 2010). The distribution of the elastic lateral seismic forces determined using a response spectrum analysis is shown in Figure 1b for the factored seismic loads and in Figure 1c when the base moment reaches the probable resistance \( M_p \). The lateral seismic force distribution corresponding to the maximum shear force at the base is shown in Figure 1d. As can be seen, the greater participation of the higher modes changes the distribution of the seismic forces, and their resultant is located at a height of \( h_1 \) (Figure 1d) from the base, which is much lower than the height of \( h_0 \) (Figure 1b and c), where the resultant of the design seismic forces is located. Since the probable resisting moment at the base must remain the same and because the resultant of the lateral seismic forces due to the higher mode effects is closer to the base, its amplitude must increase to
give the same overturning probable moment at the base. In this example, the inelastic higher mode shear amplification factor can be approximated as the ratio of \( \frac{h_0}{h_1} \), giving \( \omega_v = \frac{0.55h_w}{0.17h_w} = 3.2 \), where \( h_w \) is the height of the wall. This high shear amplification has been measured in recent experimental studies by Fatemi et al. (2020). Simplified similar reasoning can be found in Paulay and Priestley (1992).

In the case of coupled walls, the capacity design procedure requires that all coupling beams should reach their yielding capacity before the formation of plastic hinges at the base of the walls. This design procedure changes the behavior of coupled walls compared to single walls. Therefore, the shear response of the former system must be studied. Indeed, Lybas (1981) seems to be the first to experimentally observe the effects of higher modes on the shear response of coupled walls. His research showed that the second lateral mode response had a major effect on the shear demand at the base and that the contribution of higher modes to the total response increased as the strength and stiffness of coupling beams decreased. He also noted that the shear demand exceeded the values consistent with the flexural failure mechanism because of the contribution of the second mode.

**Shear amplification in cantilever walls in the New Zealand standard and the ACI code**

Blakeley et al. (1975) were the first to study the higher mode effects on the shear and flexural demand from structural walls during earthquakes. Their work led to the adoption in the New Zealand Standard (2006) of a correction factor, \( \omega_v \), to consider the elastic and inelastic effects of higher modes:

\[
\omega_v = \begin{cases} 
0.9 + N/10 & \text{for } N \leq 6 \\
1.3 + N/30 & \text{for } N > 6
\end{cases}
\]  

where \( N \) is the number of stories. As can be seen, \( \omega_v \) is limited to 1.8, which corresponds to a 15-story building. This factor was recently adopted in the 2019 ACI (2019) Code. It
was recommended in the explanatory notes of the CSA A23.3 standard published in the 1994 edition of the Concrete Association of Canada (CAC) Concrete Design Handbook (CAC, 1996). The amplification coefficient from Equation (3) is based only on the number of stories, which is a rough indicator of the fundamental period. Moreover, it neglects the flexural overstrength at the base, the seismic force reduction factor and the overstrength, which have significant influence on the shear amplification according to recent studies by Boivin and Paultre (2012a). However, the use of an amplification factor is at the moment the simplest way to account for this phenomenon.

Since the work of Blakeley et al. (1975), much research has been conducted on this subject, and an extensive list of publications is currently available. However, a consensus does not yet seem available. In addition, the correction factors proposed in North American codes were generally derived for isolated walls. Their application to coupled walls has not yet been investigated, although some design codes recommend the same amplification for walls and coupled walls. Reviews of inelastic shear amplification due to higher mode effects in cantilever shear walls have been presented in Boivin and Paultre (2012b), Rutenberg (2013) and Fatemi et al. (2020).

**Shear amplification in the CSA A23.3-14 standard**

Recent nonlinear numerical studies of cantilever walls have shown that an amplification factor, varying linearly from 1 to 2 as a function of the flexural overstrength and the fundamental period of vibration, is needed to safely estimate the seismic shear demand (Boivin and Paultre, 2012b; Dezhdar, 2012). However, the amplification factor in the CSA A23.3-14 standard has been limited to 1.5 for the following reasons (CAC, 2016): (1) The maximum shear force occurs supposedly only during a loading cycle and for a very short time for a relatively short duration earthquake excitation, while the shear is lower during other loading cycles (Yathon, 2011), and (2) the maximum shear force does not occur at the same time as the maximum rotation at the base of the wall. Thus, based on the results of Boivin and Paultre (2012b) and Dezhdar (2012), the seismic shear amplification factor in the 2014 edition of the Canadian design standard is:

\[
\omega_v = \begin{cases} 
1.0 & \text{if } T_1 \geq T_L \\
1.0 + 0.25 \left( \frac{R_o R_o}{\gamma_w} - 1 \right) & \text{if } T_1 \geq T_U
\end{cases}
\]  

(4)

in which:

\[
T_L = \begin{cases} 
0.5 & \text{for } S(0.2)/S(2.0) < 10.0 \\
0.2 & \text{for } S(0.2)/S(2.0) \geq 10.0
\end{cases}
\]  

(5)

and:

\[
T_U = \begin{cases} 
1.0 & \text{for } S(0.2)/S(2.0) < 10.0 \\
0.5 & \text{for } S(0.2)/S(2.0) \geq 10.0
\end{cases}
\]  

(6)

where \( T_L \) is the lower natural period for which there is no amplification and \( T_U \) is the upper natural period above which there is constant shear amplification. The shear amplification is assumed to vary linearly between these two periods. The limiting spectral ratios
$S(0.2)/S(2.0)<10.0$ and $S(0.2)/S(2.0)\geq 10.0$ account for specificities of ground motion characteristics in western and eastern North America. Eastern North America ground motions are characterized by a high-frequency content, and their spectral shape is steeper in the medium and high period ranges, for which higher shear amplification due to higher mode effects is observed. $\gamma_w$ is the wall nominal overstrength factor, which for all practical purposes is equal to the ratio of the nominal moment resistance, $M_n$, to the factored flexural demand, $M_f$, at the base of the wall system, that is:

$$\gamma_w = \frac{M_n}{M_f} \quad (7)$$

The precise definition of $M_n$ and $M_f$ according to the CSA A23.3-14 will be given in the following section for coupled walls.

**Methodology**

The main objective of this research is to study the seismic shear amplification in coupled shear walls when using a capacity design approach. A parametric study was therefore carried out to identify and quantify the influence of selected parameters on the shear demand. The following methodology was adopted:

1. Select studied parameters based on previous study results;
2. Select ground motions according to the most recent methods;
3. Design the walls and coupling beams for each case study;
4. Compare the shear demands obtained from the design values and NLTHA;
5. Obtain a shear amplification factor that can be used for coupled wall systems.

**Studied parameters**

The selection of the parameters was based on isolated wall results from previous works (Boivin and Paultre, 2012a; Dezhdar, 2012) and on coupled wall analyses where it was noted that the coupling beams may influence the shear response (Pennucci et al., 2015). The parameters studied in this research are listed in the following:

1. Fundamental period of vibration, $T_1$;
2. Degree of coupling (DOC) (defined below);
3. Flexural overstrength at the base, $\gamma_w$;
4. Shear design method; and
5. Geographical location of the structures.

The fundamental period corresponds to the first elastic period of numerical models according to the requirements of NBCC-15 and CSA A23.3-14. In these models, the section properties are reduced to consider concrete cracking. The geometries of these models were chosen such that the fundamental period calculated from the finite element analysis was between 1.0 and 2.0 times the empirical period, $T_u$, recommended by NBCC-15 and ASCE 7-16 for structural wall systems and defined as:

$$T_u = 0.05h_n^{3/4} \quad (8)$$
where $h_n$ is the height of the structure in meters.

The degree of coupling (DOC) is defined as the ratio of the base overturning moment caused by axial forces in the wall piers and resulting from the shear transmitted by the coupling beams to the total factored seismic base overturning moment:

$$\text{DOC} = \frac{P_{\text{cg}}}{M_f}$$  \hfill (9)

where

$$M_f = M_1 + M_2 + P_{\text{cg}}$$ \hfill (10)

where $P$ is the axial tensile or compression force at the wall pier base resulting from the vertical shear force transferred by the coupling beams that are induced by the lateral seismic forces, $\ell_{cg}$ is the distance between the wall centers of gravity, and $M_1$ and $M_2$ are the bending moments at the base of each wall pier due to lateral seismic forces. $P$, $M_1$, and $M_2$ are obtained from elastic analysis using reduced section properties for the coupling beams as recommended by CSA A23.3:

$$A_{ve} = \begin{cases} 0.15A_g & \text{for conventionally reinforced coupling beams} \\ 0.45A_g & \text{for diagonally reinforced coupling beams} \end{cases}$$ \hfill (11)

and

$$I_e = \begin{cases} 0.40I_g & \text{for conventionally reinforced coupling beams} \\ 0.25I_g & \text{for diagonally reinforced coupling beams} \end{cases}$$ \hfill (12)

where $A_{ve}$ and $I_e$ are the effective shear cross-section area and effective moment of inertia, and $A_g$ and $I_g$ are the corresponding gross properties of coupling beams. For the walls, the following properties are used:

$$A_{xe} = \alpha_w A_g; \quad I_e = \alpha_w I_g$$ \hfill (13)

where $A_{xe}$ is the effective axial cross-sectional area and $I_e$ is the effective moment of inertia of the walls, and

$$0.5 \leq \alpha_w = 1.0 - 0.35 \left( \frac{R_d R_o}{\gamma_w} - 1.0 \right) \leq 1.0$$ \hfill (14)

The parameters of Equation (9) are illustrated in Figure 2. The wall piers are called compression or traction walls depending on whether the shear transmitted by the coupling beams increases or decreases the compression force due to the gravity loads acting on each wall. Wall systems with a DOC less than 66% are defined as partially coupled walls, while the term coupled walls is used to describe those with a DOC greater than 66% in the NBCC-15 and the CSA A23.3-14 standard (see Table 1). In this research program, the DOC was adjusted by modifying the slenderness of the walls and the height-to-length ratio of the coupling beams and was assessed using the simplified equivalent seismic static (triangular) lateral forces distribution as recommended by the NBCC-15.
Two regions of Canada were selected to study the effect of the frequency content on the seismic shear demand. For western North America, the city of Vancouver was chosen because it is the Canadian city with the highest seismic risk. For eastern North America, the city of Baie-Saint-Paul was chosen because of its significant seismic hazard, allowing greater control of $g_w$, the high-frequency content of earthquakes in the area, which is characteristic of cities in eastern Canada and their spectral acceleration values, which are comparable to those of Vancouver. Notably, flexural design of ductile walls with $T_1 \geq 1.0 \text{s}$ is usually controlled by minimum reinforcement in major cities in eastern Canada, such as Montreal, because of lower seismic hazard.

The walls were designed for type D soil conditions with average shear wave velocities in the top 30 m of $180 < V_{S30} < 360 \text{ m/s}$, as defined in the NBCC-15. This choice allows better control of the nominal flexural overstrength at the base, $\gamma_w$, because the spectral values in the low-frequency range of the UHS are generally amplified for this soil type. This parameter was controlled by varying the reinforcement ratio. The decision was made to limit the study to rectangular and symmetrical sections relative to the SFRS’s center of gravity, which is halfway between the two symmetric wall piers. The list of studied cases is reported in Table 2. For western Canada, 164 models were built, while for the eastern Canada region, analyses on 130 models could be performed. Each of them was subjected to a series of 40 ground motions, which resulted in a total of 11760 NLTHAs for partially coupled and coupled wall systems.

**Record selection**

The NBCC-15 UHS has often been used as a target spectrum for record selection. These spectra are, however, too conservative for record selection because they represent the envelope of the spectral responses at all periods with a 2% probability of exceedance in 50 years.
Thus, they do not represent the spectra of any single seismic event (Baker, 2011). Furthermore, the variability of the response is reduced when UHS-compatible records are used in NLTHA, and the randomness of a seismic response may not be well represented (Dezhdar and Adebar, 2015).

An interesting alternative, proposed by Baker (2011), is to select ground motions compatible with the conditional spectrum. These spectra consider the variability and uncertainty of earthquakes. They are more representative of real earthquakes that produce a spectral acceleration $S_a$ at a target period, whose amplitude can be chosen such that the probability of exceedance is 2% in 50 years, while the mean response for the other periods is lower or higher. To build these spectra, the fundamental period is commonly used as the conditioning period. Dezhdar and Adebar (2015) have shown that this choice is not always adequate depending on the response sought. In fact, the maximum shear response will be observed if the conditional period is chosen to be the second period of vibration because the shear response is more influenced by higher modes than by the fundamental one, whose period tends to lengthen during a strong earthquake. Thus, the target period for structures with more than 10 stories was chosen to be the second elastic period of vibration and the fundamental period otherwise. This approach has been shown to maximize the shear demand.

The first step in the construction of the conditional spectrum is to identify the seismic scenario with mean magnitude/distance ($M/R$) characteristics that contribute to the hazard at the conditional period. To this end, a disaggregation analysis of the seismic hazard was performed with the open-source software OpenQuake (GEM, 2015). The $M/R$ scenario allows the calculation of a median spectrum using the appropriate ground motion prediction equation (GMPE). For western Canada, the NGA-West2 GMPE (Boore et al., 2014) was used to calculate the median spectrum and the corresponding standard deviation. For eastern Canada, the GMPEs developed by Atkinson and Adams (2013) were used to construct the predicted spectra. For the chosen soil type, the NGA-West2 GMPE can consider amplification of the signal by specifying the average shear wave velocity in the first 30 m. For eastern Canada, the soil amplification factors provided by the NBCC-15 were used to consider the soil condition in the GMPEs.

### Table 2. Varying parameter values of coupled walls for the parametric study

| Wall type               | Number of stories, $N$ | $T_1$, s | Degree of coupling, % | $\gamma_w$ |
|-------------------------|------------------------|----------|-----------------------|------------|
| Partially coupled walls | 3                      | 0.3      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
|                         | 5                      | 0.5      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
|                         | 10                     | 1.0      | 20, 30, 40, 50, 60    | 2.0, 2.5, 3.0, 4.0 |
|                         | 10                     | 1.5      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
|                         | 15                     | 2.0      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
|                         | 20                     | 2.5      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
|                         | 25                     | 3.0      | 30, 40, 50, 60        | 2.0, 2.5, 3.0, 4.0 |
| Coupled walls           | 5                      | 0.5      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |
|                         | 10                     | 1.0      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |
|                         | 10                     | 1.5      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |
|                         | 15                     | 2.0      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |
|                         | 20                     | 2.5      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |
|                         | 25                     | 3.0      | 70, 75                | 2.0, 2.5, 3.0, 4.0 |

aData not available for eastern Canada.
For each of the seven selected conditional periods, a set of 40 historical records were selected and scaled for Western Canada and Eastern Canada sites, resulting in $40 \times 72 = 560$ historical records selected. Figure 3 shows the 40 response spectra for historical records conditioned on a 0.5 s period for Eastern Canada and on a 1.5 s period for Western Canada.

Shear design of coupled walls

As mentioned, the design of coupled shear walls is analogous to the strong-column weak-beam procedure in frames in which yielding of all coupling beams precedes the formation of plastic hinges at the base of the walls as shown in Figure 4a. The elastic distribution of the shear forces in coupling beams over the height of coupled wall systems is typical, with very large shear forces in the lower one-third to one-half of height of the system, with a

![Figure 3](image1.png)

**Figure 3.** Uniform hazard spectrum (UHS), conditional mean spectrum (CMS), and response spectra for historical records conditioned on (a) a 0.5 s period for Baie-Saint-Paul and (b) a 1.5 s period for Vancouver, bounded by ±2 standard deviation ($\sigma_{\text{cond}}$).

![Figure 4](image2.png)

**Figure 4.** Shear design of coupling beams in a coupled wall system: (a) energy dissipating mechanism and (b) shear force distribution in coupling beams and redistribution of shear forces over the height of the structure.

For each of the seven selected conditional periods, a set of 40 historical records were selected and scaled for Western Canada and Eastern Canada sites, resulting in $40 \times 72 = 560$ historical records selected. Figure 3 shows the 40 response spectra for historical records conditioned on a 0.5 s period for Eastern Canada and on a 1.5 s period for Western Canada.

**Shear design of coupled walls**

As mentioned, the design of coupled shear walls is analogous to the strong-column weak-beam procedure in frames in which yielding of all coupling beams precedes the formation of plastic hinges at the base of the walls as shown in Figure 4a. The elastic distribution of the shear forces in coupling beams over the height of coupled wall systems is typical, with very large shear forces in the lower one-third to one-half of height of the system, with a
rapid reduction in the shear force amplitude toward the top of the system as shown by the solid line in Figure 4b. Two types of coupling beam reinforcements are recommended to resist the shear stresses in the coupling beams. For $\text{DOC} \geq 0.66$, diagonal shear reinforcement as proposed by Paulay (1969; Paulay and Binney, 1974) was used as recommended by the CSA A23.3 standard, while conventional flexure and shear reinforcements were used otherwise. It is recommended by Paulay and Priestley (1992) and by the CSA A23.3-19 to use redistribution of the shear forces in coupling beams designed with diagonal reinforcement because of their inherent ductility (Paulay and Priestley, 1992) to achieve a more economical design because a large number of beams can have the same reinforcement over the height of the structure as shown by the broken lines and shaded area in Figure 4b. Reduction of shear forces not exceeding 20% of the vertical shear forces predicted in the coupling beam is allowed while keeping the resultant vertical shear force of all coupling beams larger or equal to the elastic vertical shear force resultant so that the total axial load introduced in the walls is not reduced. Because of the automatic design used for very many buildings, only manual redistribution was used in some cases. To guarantee the safety of the coupled walls, the design envelope for shear must be greater than the demand over the entire height of the wall system, similar to cantilever walls, as expressed in Equation (4) with amplification factor $\omega_v$ taken as 1, because, as mentioned previously, no shear amplification is recommended in the current CSA A23.3-14 standard for coupled wall systems.

Because the coupling action of structural walls linked by properly designed slabs or shallow beams (see Figure 2a) is negligible, these walls can be considered two isolated cantilevered walls. For these systems, both walls of the symmetrically reinforced, rectangular SFRS will have the same probable flexural overstrength because the axial forces resulting from the shear transmitted by the coupling beams are low compared to the gravity loads. However, in the case of moderately coupled walls with $\text{DOC} < 0.66$ (Figure 2b) and strongly coupled walls with $\text{DOC} \geq 0.66$ (Figure 2c), two approaches are used to determine the design envelope for shear and are described in the following subsection.

**Shear design envelope for ductile partially coupled walls.** In the case of ductile partially coupled walls with $\text{DOC} < 0.66$, each wall is designed according to the capacity design method for the factored shear envelope amplified by the ratio $M_p/M_f$, where $M_f$ is the factored moment at the base of the wall with the largest probable moment $M_p$ at this location, which corresponds to the case with the largest compression force acting on the so-called compression wall. This is so because structural walls usually have a small amount of flexural reinforcement (flexural reinforcement ratio less than 1% and axial load less than $0.1\,\frac{Agf'_c}{f'_c}$) and the flexural strength increases with increasing axial load up to approximately $0.25\,\frac{Agf'_c}{f'_c}$ of a wall pier, where $Ag$ is the gross cross-sectional area of the wall pier, and $f'_c$ is the specified concrete compression strength. This method will be referred to in this paper as individual wall elements for shear design (IWESD). Hence, the shear reinforcement in both walls according to this approach is governed by the design shear envelope of the compression wall.

**Shear design envelope for ductile coupled walls.** To determine the shear design envelope of coupled wall systems with a high DOC, the approach recommended by Paulay and Priestley (1992) in which, according to the capacity design method, the factored shear envelope is amplified by the ratio $M_p/M_f$, where, in this case, $M_f$ is the factored moment and $M_p$ is the probable moment resistance, both evaluated at the wall base of the entire wall system which, for a two-coupled walls is the sum of the probable resistance of both
walls $M_{1p}$ and $M_{2p}$ and the couple component $P_p \ell_{cg}$ resulting from the probable axial forces in the wall piers, as described in the next section. This method is referred to as single wall element for shear design (SWESD).

**Determination of the nominal and probable flexural overstrengths**

The probable flexural overstrength is used to determine the amplified design shear envelope according to capacity design procedures, while the nominal flexural overstrength is used to further increase the shear envelope to account for the shear amplification due to inelastic higher mode effects. It indicates the capacity reserve that the walls possess compared to the factored moment resistance required to resist the factored design forces. In the CSA A23.3-14 standard, the axial force ($P_n$) arising from the development of the nominal shear capacity of the coupling beams is included in the calculation of the nominal bending moment resistance of the wall system according to

$$M_n = M_{1n} + M_{2n} + P_n \ell_{cg}$$

(15)

where $M_{1n}$ and $M_{2n}$ are the nominal flexural resistances of the wall piers, and $P_n$ is the earthquake-induced nominal axial force resulting from the interaction of the coupling beams and the walls, all evaluated at the base. When calculating $P_n$, one must consider that all coupling beams in a multistory building will not reach their nominal resistance at the same time (Moehle, 2015; Paulay and Priestley, 1992). The CSA A23.3-14 standard recommends determining $P_n$ by weighting the axial design force at the base, $P_f$, obtained from a combination of spectral responses according to the following relationship:

$$P_n = \left( \frac{\sum_{i=1}^{N} V_{ni}^{cb}}{\sum_{i=1}^{N} V_{hi}^{cb}} \right) P_f$$

(16)

where $N$ is the number of stories, $V_{ni}^{cb}$ is the nominal shear resistance of the coupling beam at the level $i$, and $V_{hi}^{cb}$ is the corresponding factored shear force in the coupling beam at the level $i$

The determination of the probable moment resistance, $M_p$, of a coupled walls system is similar to the calculation in Equation (15), replacing $n$ by $p$, with the nominal strength of the coupling beam, $V_{ni}^{cb}$ in Equation (16), increased by a factor of 1.25 and the strength of the vertical reinforcement increased by a factor of 1.25 when calculating $M_{1p}$ and $M_{2p}$, the probable moment resistance of each wall.

**Modeling for inelastic analyses**

Nonlinear analyses were performed with OpenSees (OS) software (McKenna et al., 2010). The walls were modeled with the multilayered beam-column elements as shown in Figure 5c. The number of degrees of freedom is greatly reduced with this model, which considerably reduces the computation time compared to full finite element modeling. The cross section of this line element is subdivided into multiple layers of concrete and steel, for which a material behavioral law is specified. The uniaxial concrete behavior was modeled with the Concrete06 model Popovics (1973), while the reinforcement was modeled
with the Steel02 law, which considers strain hardening and the Bauschinger effect. The fiber elements neglect the interaction between the shear, flexure, and axial load because their formulation is based on Euler–Bernoulli beam theory. Thus, the shear deformations were considered uncoupled with an elastic behavior. P-delta effects were considered through a corotational transformation. The walls were modeled with one force-based beam column element per story with five integration points. The mass of the building was concentrated on each floor.

The coupling beams were modeled with linear beam elements with concentrated zero-length hysteretic hinges at both ends since yielding is expected at the ends of the coupling beams. The initial stiffness of these hinges was calculated from an effective inertia using the expression proposed by Son Vu et al. (2014) for conventionally and diagonally reinforced coupling beams. A chord rotation model, according to ASCE 41 (ASCE, 2014), was used to estimate the end yield moment as a function of the yield rotation, resulting in an initial stiffness,

\[ k_0 = \frac{6E_cI_e}{\ell_u} \]  

Figure 5. Modeling of coupled walls in OpenSees: (a) wall system, (b) finite element model, (c) layered beam-column element, (d) Concrete06 behavioral model, (e) Steel02 behavioral model, and (f) backbone of zero-length rotational spring element at the end of coupling beams.
where $I_e = \kappa I_g$ is the effective moment of inertia of the coupling beam, $I_g$ is the moment of inertia of the gross sectional area, and $\kappa$ is a factor that depends on the geometry and detailing of the reinforcement of coupling beams. Son Vu et al. (2014) recommend $\kappa = 0.671$ for conventional shear reinforcement and $\kappa = 0.651$ for diagonal shear reinforcement, in which $\ell_u$ is the clear length of the coupling beam, $d$ is the effective depth of the coupling beam, $f_c'$ is the concrete strength, $\rho_s$ is the longitudinal reinforcement ratio, $\rho_v$ is the transverse reinforcement ratio, and $\rho_d$ is the diagonal reinforcement ratio.

The moment-rotation behavior was described with the modified Ibarra-Medina-Krawinkler deterioration model with a bilinear hysteretic response (Ibarra et al., 2005; Lignos and Krawinkler, 2011). The parameters $a$, $b$ and $c$ describing the backbone curve shown in Figure 5f were taken from ASCE 41-13 (ASCE, 2014) and are presented in Table 3. Rigid end zones were used to connect the hysteretic hinges to wall centers of gravity.

Damping ratios between 1% and 2% were measured on undamaged reinforced concrete structural walls (Boroschek and Yáñez, 2000). ATC-72-1 (2010) recommends the use of Rayleigh damping for nonlinear dynamic analyses with a damping ratio between 2% and 5%. However, recent numerical analyses have shown that the higher mode effects could be hidden when a 5% damping ratio is considered (Boivin and Paultre, 2010). Thus,

### Table 3. Nonlinear parameters for modeling of coupling beams ASCE (2014).

| Type of coupling beam reinforcement | Chord rotation |
|-------------------------------------|----------------|
|                                     | a   | b   | c   |
| Conventional                        | 0.020 | 0.040 | 0.50 |
| Diagonal                            | 0.030 | 0.050 | 0.80 |

The moment-rotation behavior was described with the modified Ibarra-Medina-Krawinkler deterioration model with a bilinear hysteretic response (Ibarra et al., 2005; Lignos and Krawinkler, 2011). The parameters $a$, $b$ and $c$ describing the backbone curve shown in Figure 5f were taken from ASCE 41-13 (ASCE, 2014) and are presented in Table 3. Rigid end zones were used to connect the hysteretic hinges to wall centers of gravity.

In the nonlinear response, energy dissipation is primarily due to hysteretic damping from the inelastic deformation of the structure. However, other mechanisms, such as internal damping, are present when structures exhibit dynamic behavior. These mechanisms are difficult to quantify and are generally modeled with Rayleigh damping. In this approach, the damping matrix is chosen to be proportional to the mass and stiffness matrices. The fundamental period lengthening will lead to an increase in the modal damping ratio and cause unrealistically large damping forces compared to the internal forces in the equation of motion (Hall, 2006). To limit these problems, the updated tangent stiffness matrix should be used at each step of the time-history analyses to define the stiffness proportional term for a system when no abrupt changes in stiffness, such as the systems analyzed in this research program, occur (Charney, 2008).

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Dynamic analysis results

The seismic shear amplification factors at the base shown in Figures 6 to 8 were calculated from the average results of 40 ground motions compatible with the conditional spectra. They represent the ratio between the maximum shear demand ($V_{de}$) observed during the NLTHA, and the design shear ($V_{dp}$) is defined in Equation (2).

For partially coupled walls designed with the IWESD approach (see Figure 6), $V_{de}$ corresponds to the maximum shear observed during the NLTHA in one wall at the base. In the case of ductile coupled walls (DOC ≥ 66%) designed with the SWESD method (see Figure 7), the seismic amplification factor corresponds to the ratio of the maximum total shear demand at the base to the sum of the design shear forces in the walls.

As stated earlier, shear design envelopes differ between the IWESD and SWESD approaches. It is complex to define a specific DOC value for which a design method should be favored. Figure 8 shows the influence of the design approach on the seismic shear amplification factor for partially coupled walls with a DOC of 50% located in Vancouver. As can be seen, both methods tend to generally underestimate the shear demand, while the IWESD method could overestimate the shear force in systems with large overstrength values. Both techniques could be used for design if an appropriate correction factor is considered to amplify the design shear demand when redistribution of the shear forces between the two walls is allowed, as the SWESD method is based on this assumption.

Influence of flexural overstrength at the base

The results from these nonlinear analyses show that the shear amplification decreases with increasing nominal flexural overstrength at the base. Because design forces are higher as the flexural capacity is increased, this leads to a reduction in the shear amplification factor. This influence of the nominal flexural overstrength at the base is observed independent of the period for every DOC value studied. Furthermore, ground motions compatible with the conditional spectra conditioned at frequencies higher than the fundamental one led to a higher shear demand, while the flexural demand at the base was reduced. Thus, the flexural demand at the base predicted by NLTHAs was generally lower than the probable flexural resistance from the CSA A23.3-14 standard for high flexural overstrength values.

Influence of the fundamental period

Seismic shear amplification is also a function of the fundamental period of vibration. Its influence on the shear demand of coupled and partially coupled walls is more significant when the period increases from 0.2 s to 1.0 s, whereas for periods greater than 1.0 s, the results show a constant amplification factor for the two studied regions (see Figures 6 and 7). Furthermore, the influence of $T_1$ is greater in Eastern Canada, which is characterized by high-frequency content earthquakes. The design spectra in this region exhibit a larger ratio between the second and first mode spectral responses, $S(T_2)/S(T_1)$. Hence, higher amplification values are expected since the demand is strongly influenced by the
higher modes. The spectrum shape is currently not considered in the calculation of the seismic amplification factor for shear walls in the CSA A23.3-14 standard. It is accounted for in Eurocode 8 (2005), which recommends a shear amplification factor based on the ductility level, the flexural overstrength, and the response spectral ratio.

**Figure 6.** Seismic shear amplification factor for partially coupled shear walls for different DOC values for (a) Western Canada and (b) Eastern Canada.
Influence of the degree of coupling

The degree of coupling is the third parameter that showed the highest influence on the shear amplification. As observed in Figure 6, the dynamic amplification decreases with the DOC. For low DOC values, the results tend toward those of isolated shear walls, while for DOCs near 60%, the actual provisions of capacity design in CSA A23.3-14 are adequate for estimating shear demand for $\gamma_w = 3.0$.

Strongly coupled walls are designed to exhibit similar behavior to isolated walls with openings. These systems, designed according to the SWESD method as part of this study, will present a lower shear demand than shear walls. The coupling beams are expected to contribute in a more substantial way to the hysteretic damping in coupled wall systems. In addition, yielding of coupling beams leads to a greater increase in higher mode periods, which contributes to a reduction in the shear demand of these modes.

Proposed modification of the capacity design in CSA A23.3

The amplification of shear forces by a factor $\omega_v$ (see Equation (4)) has been preferred since the 2014 edition of the CSA A23.3 standard. It is also the method used by the New Zealand standard and the ACI code. Compared to several proposed methods found in the literature, this method is easily applied and produces an adequate estimate of the shear demand of RC structural walls. In addition, the amplification factor allows one to quantify the higher mode effects neglected in the response spectrum analysis. Hence, a simple
The approach is to apply a modification to the correction factor proposed by the CSA standard to reflect the results obtained from nonlinear analyses and to include the DOC as a parameter of the dynamic shear amplification factor. The results of this study also suggest that an upper limit on $v_v$ of approximately $2^\ast$ is appropriate to correctly capture the high shear demand in partially coupled walls with low nominal overstrength values $\gamma_w$ and in cantilever shear walls. The following changes are proposed to the CSA A23.3 requirements:

$$\omega_v = \begin{cases} 
1.0 & \text{if } T_1 \leq T_L \\
1.0 + \eta \cdot 0.25 \left( \frac{R_d R_o}{\gamma_w} - 1 \right) & \text{if } T_1 \geq T_U
\end{cases}$$

where

$$\eta = \begin{cases} 
2.5(1.0 - 1.2 \times \text{DOC}) & \text{for partially coupled walls}^* \\
1.0 & \text{for strongly coupled walls}^†
\end{cases}$$

is a correction factor introduced to consider the influence of the DOC on the seismic shear amplification in the expression recommended in the CSA A23.3-14 standard, $T_L$ is taken as 0.2 s, and $T_U$ is as given in Equation (5). A linear interpolation is used for the fundamental period between $T_L$ and $T_U$. Equations (20) and (21) are illustrated in Figure 9 together with the numerical values obtained for Vancouver. It can be observed that the $\omega_v$ factor for partially coupled walls decreases as the DOC decreases, and the nominal flexural overstrength increases to a lower limit of 1.0.

The proposed $\eta$ factor requires that coupled and partially coupled walls are designed according to the SWESD and IWESD methods, respectively. For coupled walls, the greater ability to dissipate the energy of these systems is incorporated by taking $\eta$ equal to 1.0. Although the DOC has a slight influence on $\omega_v$ in strongly coupled walls, the slenderness of the coupling beams has no significant influence on the DOC at these high values.

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*Partially coupled walls designed with the IWESD method.
†Fully coupled walls designed with the SWESD method.
Consequently, the DOC is almost constant, as its value will generally be between 0.70 and 0.80. For this reason, the amplification value is based on a DOC of 75% to simplify Equation (21). The proposed $\eta$ factor was calibrated on the results found for Western Canada (Vancouver). This approach may lead to an underestimation of shear demand for the high seismic hazard regions of Eastern Canada. However, these regions are less densely populated and unlikely to have high-rise buildings. It should be emphasized that the proposed Equations (20) and (21) do not reduce to Equations (4) and (5) when $\text{DOC} = 0$ for cantilever walls but give larger amplification values that are consistent with findings from Boivin and Paultre (2012b) and Dezhdar (2012) and supported by experimental results obtained by Fatemi et al. (2020). Hence, for a ductile cantilever wall with $R_dR_o = 3.5 \times 1.6 = 5.6$ and a minimum overstrength factor $\gamma_w = 2.0$, the shear amplification factor is $\omega_v = 2.1$. From findings of this research program and results from previous research programs on cantilever walls, a maximum shear amplification of at least 2 is recommended. This maximum value is somewhat higher that the maximum value of 1.8 recommended in the New Zealand standard and the ACI code.
Discussion

Nonlinear analyses have shown that the seismic shear amplification may be higher in eastern Canada, which is characterized by earthquakes with a high-frequency content. The decision was made to neglect the influence of the design spectrum shape in the above proposition since only two cities have been studied, and the seismic risk is low in eastern regions with high seismic hazard. This parameter could have been expressed as the ratio $S(T_2)/S(T_1)$, with higher $\omega_v$ values as this ratio increases. Further investigations on more localities with different spectral shapes are, however, required.

The proposed design method was developed for ductile, rectangular, and isolated coupled walls. Current CSA standard provisions extend the use of the amplification factor to moderately ductile shear walls. For the case of moderately ductile coupled walls, fewer inelastic deformations are expected in the coupling beams since they are designed for higher loads. This design will lead to a reduction of the hysteretic damping, and elongation of higher mode periods is expected to be less important. Thus, the DOC may have a lesser effect on moderately ductile systems, and the proposed method may underestimate the shear demand. However, moderately ductile coupled walls generally present lower $\gamma_v$ values, which result in higher $\omega_v$ values according to Equation (20). For these reasons, the authors suggest extending the method proposed in this article to moderately ductile, coupled, and partially coupled walls according to NBCC 2015.

The $\eta$ factors proposed in Equation (21) are based on the assumption that the SWESD and IWESD methods are used to design coupled and partially coupled walls, respectively. Because of the two design approaches, there is a discontinuity in the respective $\eta$ factor. The shear amplification is lower for coupled walls than for cantilever walls for both method and reduces with an increase of the DOC. The reduction is a continuous function for partially coupled walls when using the IWESD method for calculating the capacity design shear envelope. In the IWESD method, the significant reduction in the shear amplification factor is explained by the rapid increase in the probable overstrength $M_p/M_f$ with the degree of coupling. Indeed, the increase in DOC necessarily leads to an increase in the compression force in the compressed wall and thus in the probable moment, $M_p$, at the same time as a reduction in the flexural demand at the base of the walls, that is, for a DOC of 60%, only 20% of the bending moment at the base is taken up in each of two symmetric walls. Thus, strongly coupled walls (DOC $\geq 0.66$) designed with the IWESD method will have a high probable flexural overstrength and the value of the seismic amplification will tend toward 1.0. Although the method overestimates the $M_p/M_f$ ratio observed in NLTHA, it will lead to a safe estimate of shear in strongly coupled walls. However, the proposed provisions cannot be applied to partially coupled wall systems designed according to the SWESD approach because this may underestimate the maximum shear forces expected during an earthquake, as confirmed in Figure 8.

Conclusion

This research program focused on the seismic shear demand for reinforced concrete, ductile, partially, and fully coupled walls. To the authors’ knowledge, no parametric study of this extent had been conducted on this subject. Shear amplification due to higher mode effects is now well known for cantilevered RC walls and has been the subject of several numerical and experimental studies. Since 2014, the CSA A23.3 standard has recommended a simple way to amplify shear forces to account for inelastic higher mode effects. However, this factor only applies to isolated single walls, and no indication is currently
available for the quantification of higher mode amplification in ductile coupled walls. Recent research has shown lower amplification of the shear demand for coupled and partially coupled walls, which is attributable to the presence of coupling beams in each story. The results of this research have shown that the shear amplification due to higher mode effects is substantial in coupled and partially coupled walls and must therefore be considered. The influence of the studied parameters was also quantified, and changes to the capacity design of coupled walls in the CSA A23.3 are proposed. These changes can be incorporated into other North American codes. The proposed changes are based on the results of a parametric study, whose important conclusions are summarized in the following:

1. Contrary to what is believed, a shear amplification occurs in coupled shear walls due to inelastic higher mode effects.
2. This shear amplification factor decreases with increasing nominal flexural over-strength, $\gamma_w = \frac{M_n}{M_f}$, and the degree of coupling, both evaluated at the base of the walls.
3. The higher mode effects on the shear response increase linearly up to a value of the fundamental period $T_1$ greater than 1.0 s and remain constant thereafter.
4. The shear amplification is more important for the eastern North America region, where earthquakes are characterized by a high-frequency content. However, more analyses are required with different hazards to specify a spectral shape-related amplification factor.
5. The probable shear at the base is strongly dependent on the shear design approach used, and thus, a different shear amplification factor due to inelastic higher mode effects has to be employed. This research suggest a limit on the DOC when the shear design should be based on the most heavily loaded wall (IWESD) ($DOC < 0.66$), or the system of walls as a whole (SWESD) ($DOC \geq 0.66$) should be used in the shear design with their appropriate correction factor based on the degree of coupling. However, the results have shown the necessity of clarifying the recommended design method in the CSA A23.3 standard.

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**Appendix I**

**Notation**

The following symbols are used in this article:

- \( a, b, c \): parameters describing backbone of hysteretic spring;
- \( A_g \): wall gross sectional area;
- \( d \): effective depth of reinforced concrete section;
- \( k_0 \): initial stiffness of coupling beam;
- \( f_c' \): specified compressive strength of concrete;
- \( f_y \): specified reinforcement yield strength;
- \( h_n \): height of structure in meters;
- \( I_e \): effective moment of inertia of a coupling beam;
- \( I_g \): moment of inertia of the gross sectional area of a coupling beam;
- \( \ell_{cg} \): distance between wall centers of gravity;
- \( \ell_u \): clear span of coupling beams;
- \( \ell_w \): length of walls;
- \( M_f \): factored design moment;
$M_n$, $M_p$ nominal and probable base moment capacity;
$N$ number of stories of building;
$P_f$ factored design axial forces in walls from coupling beam;
$P_n$ nominal axial forces in walls from coupling beams;
$R_d$ ductility-related seismic force reduction factor;
$R_o$ overstrength-related seismic force reduction factor;
$S_o$ spectral acceleration;
$S(T_i)$ design spectral acceleration at the period $T_i$;
$T$ period of vibration;
$T_d$ empirical fundamental period from NBCC-15;
$T_i$ period of vibration of mode $i$;
$T_L$ lower limit of the fundamental period for shear amplification;
$T_U$ upper limit of the fundamental period for shear amplification;
$T^*$ target period of conditional spectrum;
$T_f$ fundamental mode period;
$T_2$ second mode period;
$V_{de}$ shear demand at the base from NLTHA;
$V_e$ elastic base shear from response spectrum analysis;
$V_{fb}^i$ factored shear design force of the coupling beam at story $i$;
$V_{ni}^i$ nominal shear capacity of the coupling beam at story $i$;
$V_{dp}$ probable design shear force in walls;
$V_{dp}^i$ amplified design shear demand at the base of walls;
$V_{s30}$ average shear wave velocities in the top 30 m of soil;
$\gamma_{w}$ nominal flexural overstrength at the base $M_n/M_f$;
$\eta$ proposed correction factor for shear amplification;
$\kappa$ effective moment of inertia of the coupling beam $I_e/I_g$;
$\phi_s$, $\phi_c$ partial safety factor for steel reinforcement and concrete;
$\mu$ design displacement ductility level (New Zealand standard);
$\rho_s$ longitudinal reinforcement ratio;
$\rho_v$ transverse reinforcement ratio;
$\rho_d$ diagonal reinforcement ratio;
$\omega_v$ dynamic shear amplification factor.