The 11th Canadian Conference on Earthquake Engineering

Canadian Association for Earthquake Engineering

SEISMIC EVALUATION OF RC SHEAR WALL BUILDINGS SUBJECTED TO HIGH FREQUENCY CONTENT EARTHQUAKES

Hieu LUU
Post-Doctoral Fellow, Department of Civil, Geological and Mining Engineering, Polytechnique Montréal, Canada
guang-hieu.luu@polymtl.ca

Pierre LÉGER
Professor, Department of Civil, Geological and Mining Engineering, Polytechnique Montréal, Canada
pierre.leger@polymtl.ca

Robert TREMBLAY
Professor, Department of Civil, Geological and Mining Engineering, Polytechnique Montréal, Canada
robert.tremblay@polymtl.ca

ABSTRACT: The objective of this paper is to apply ASCE/SEI 41-13, latest guidelines for "Seismic Evaluation and Retrofit of Existing Buildings", in the Canadian codes context for reinforced concrete shear wall buildings subjected to high frequency content eastern North America (ENA) ground motions. The seismic performance of a 10-storey shear wall building located in Montreal and designed using past and current Canadian national building and reinforced concrete codes is assessed using ASCE/SEI 41-13. In the first design approach (1), amplification of shear forces and moments due to nonlinear higher mode effects (HMEs) is ignored. The second wall design methodology explicitly considers HMEs using shear amplification factors ϖ_v from A23.3-2014 (21.5.2.2.7) and moment amplification factors with suitable envelopes along the wall height. The progressive analysis procedure prescribed in ASCE/SEI 41-13 is investigated, including: (a) linear static, (b) linear dynamic, (c) nonlinear static and (d) nonlinear dynamic analyses. The results indicate that static procedures provide different conclusions relative to the building performance compared to dynamic procedures because of significant HMEs in ENA regions. Both the Immediate Occupancy and Life Safety performance levels are achieved and plastic deformations were constrained at the base of the walls when using the design approach considering HMEs. . On the other hand, the design approach without consideration of HMEs is unsafe because of shear failure.

1. Introduction

ASCE/SEI 41-13 (ASCE 2013), "Seismic Rehabilitation of Existing Buildings", provides guidelines to perform seismic evaluation of existing buildings. ASCE/SEI 41-13 defines the damage threshold to achieve immediate occupancy (IO), life safety (LS), and collapse prevention (CP) performance levels. ASCE/SEI 41-13 has been widely used for seismic evaluation of existing reinforced concrete (RC) shear wall buildings in the civil engineering community and by some researchers (Birely et al. 2014, Hagen 2012, Gonzales and Almansa 2012). However, these studies are for shear walls located in western North America and designed according to American codes (ACI 2011). Shear walls designed according to Canadian building and concrete codes are different from those from American code. In addition, the ground motions in eastern North America are inherently rich in high frequencies, of the order of 5 to 10 Hz, which are coinciding with the frequencies of high vibration modes of RC shear walls. Therefore, there is still a question about the adoption of ASCE/SEI 41-13 to seismically evaluate RC shear walls located in eastern North America and designed according to Canadian building and concrete codes.

This paper presents the implementation of ASCE/SEI 41-13 for RC shear wall buildings subjected to high frequency content Eastern North America (ENA) ground motions. It is conducted by using ASCE/SEI 41-13 to assess the seismic performance of two alternative design approaches using Canadian building and RC codes for an existing moderately ductile (MD) 10-storey RC shear wall building located in Montreal. The two design approaches are based on : (1) previous (labelled as CW-1977, in which CW stands for conventional wall) and current (labelled as MD-2014 in which MD stands for moderately ductile) versions of Canadian building and concrete codes. Herein, (1) CW-1977 refers to NBCC 1977 (NRCC 1977) and CSA A23.3 1973 (CSA 1973); and (2) MD-2014 refers to NBCC 2010 (NRCC 2010) and CSA A23.3-14 (CSA 2014).

2. Studied Building

2.1. Building descriptions

Two alternative design approaches of a 10-storey RC residential building adapted from Luu (2014) have been selected for this study (Fig. 1a). The building is located in Montreal, Quebec, Canada and has a total height of 27.32 m. The first design approach, (1) CW-1977, is the existing design of the building (Figs 1a, b) which was designed according to NBCC 1977 and CSA A23.3-1973. The other (2) is a new design MD-2014 (Fig.1c), which is implemented in the plan view of the existing building, following NBCC 2010 and CSA-A23.3-14 for moderately ductile (MD) RC shear walls. A ductility-related force reduction factor Rd=2.0 and overstrength-related force modification factor R_0=1.4 are used for MD shear walls. This new design is conducted to respect the architectural decisions related to the core locations in the centre of the building designed in 1977. However, the analysis indicates that it is impossible to meet the NBCC 2010 requirements for drift (2.5%) and shear force with the two existing cores. This is mainly because of the large irregular torsional effect in the existing building (B=2.0) that appears to have not been considered adequately. Therefore, the location of two cores are kept but re-designed and four additional shear walls are added to the current plan view of the two new designs. More details about these two designs can be found in Luu (2014).

The main difference between two design approaches, CW-1977 and MD-2014, is from HMEs consideration methodologies. In CW-1977, using NBCC 1977 and CSA A23.3-1973, there is no requirement to consider inelastic HMEs. On the other hand, MD-2014 explicitly considers HMEs using shear force correction factors (clause 21.5.2.2.7 in A23.3-2014) and moment amplification factors with suitable envelopes along the wall height (clause 21.5.2.2.3 in A23.3-14). For shear force correction factor, Clause 21.5.2.2.7 in CSA A23.3-14 prescribes that the factored shear force shall be firstly increased to account for flexural overstrength (γ_w), and further increased for inelastic HMEs by a dynamic amplification factor, ϖv. The overstrength factor, γ_w, is the ratio of probable and nominal moment resistances to factored moment at base of shear wall for ductile and MD shear walls, respectively. The dynamic amplification factor, ϖ_v, depends on the fundamental lateral period of vibration of the building in the direction under consideration as follows:

$$\varpi_v = \begin{cases} 1.0 + 0.25\left(\frac{R_d R_o}{\gamma_w} - 1\right) \le 1.5 \text{ and } \ge 1.0 & \text{if } T \ge T_U \\ 1.0 & \text{if } T \le T_L \end{cases} \tag{1}$$

For T between T_L and T_U, linear interpolation shall be used, where T_L and T_U shall be determine as in Table 1.

Table 1. Proposed parameters for shear amplification factor considering inelastic HMEs in CSA A23.3-14

S(0.2)/S(2.0)	T_L(s)	T_U (s)
<8.0	0.5 s	1.0 s
>8.0	0.2 s	0.5 s

For moment envelope, Clause 21.5.2.2.3 in CSA A23.3-14 prescribes that for both ductile and MD shear walls, the bending moment above the plastic hinge region obtained from linear analysis, M_f, is amplified to prevent an inelastic response in the upper part of the walls. The amplification factor for the upper part is the ratio of the factored moment resistance (M_r) to the factored moment (M_f), both of which are calculated at the top of the plastic hinge region.

2.2. Structural models of RC shear wall buildings for EQ response analyses

Both linear and nonlinear models are used in this study. Elastic models using ETABS (CSI 2010) are employed to conduct linear analyses for the CW-1977 and MD-2014 designs. The ETABS models are also used to evaluate the performances of the two designs (CW-1977 and MD-2014). The models are developed according to the guidelines provided in NBCC 2010 for the new design and ASCE/SEI 41-13 for the evaluation of the two alternative design approaches. In ETABS models, the wall/slab elements, which are shell-type elements with a membrane and bending component, are used for walls and floor slabs. Rigid floor diaphragm constraints are used. The seismic force resisting systems are assumed to be decoupled from the slab for the storey under the ground level. The fundamental period using shear wall effective stiffness according to NBCC 2010 of the first three mode for MD-2014 design is: 1.5s (U_Y), 1.4s (U_X), and 0.9 s (T_Z); while for CW-1977, these are: 4.7s (T_Z), 2.4s (U_Y), and 1.6s (U_X). U_X, U_Y, and T_Z herein are translational modes for X and Y directions, and torsional mode around vertical Z direction.

Nonlinear time history analyses using PERFORM 3D (CSI 2013) models are used in this study. The model uses vertical fibre elements to explicitly model the nonlinear properties of the wall cross sections. For the dynamic analyses, the masses are concentrated, following a lumped-mass approach at the centroid of every floor. The rotational inertias are also considered. The values of masses, rotational inertias and location of the centroid of every floor are imported from ETABS models. The PERFORM 3D models in this study are adapted from the models used in Luu et al. (2014b).

3. Seismic Performance assessment of RC Shear Wall Building - ASCE/41-13

ASCE/SEI 41-13 (ASCE 2013), a standard published by the American Society of Civil Engineers, provides guidance for design professionals to determine whether an existing building can adequately resist seismic forces. The standard provides three performance levels to identify potential deficiencies in seismic designs: (a) Immediate Occupancy (IO): the building remains safe to occupy, and any repair is minor; (b) Life Safety (LS): the building may experience extensive damage to structural and non-structural components. Repairs may be required before the building can be reoccupied, and repair may be deemed economically impractical. The risk of the building to life safety by meeting this target building performance level is low; and (c) Collapse Prevention (CP): the building may pose a significant hazard to life safety due to failure of a non-structural component. However, the building itself does not collapse. Loss of life may be avoided (ASCE 2013).

In Canada, buildings are designed according to the Canadian building codes subjected to an earthquake that has a 2% probability of exceedance in 50 years. The target performance level of the building is LS. In this study, the building is deemed adequate and safe if the design procedure provides a LS seismic performance.

To assess a shear wall according to ASCE/SEI 41-13, we must first determine whether the inelastic deformation of walls is governed by flexure or shear under lateral loading. The identification of each category depends on the relative strength of the wall resisting mechanisms (flexural and shear strengths). Therefore, one normally identifies the dominant resistance mechanism of the wall by applying a uniform or inverted triangular lateral load distribution. Next, the internal shear force at the nominal flexural strength is calculated. The wall is considered to be controlled by shear if this value is greater than the shear strength of the component and controlled by flexure if the shear at nominal flexural strength is less than the shear strength of the component.

However, some studies have found that the seismically induced load distribution in a shear wall under earthquake loading can vary significantly (Luu et al. 2014; Ghobanirerani et al. 2012; Dezhdar and

Adebar 2012). A higher ratio between shear and flexural demands may occur at the base for the equivalent static force procedure with an inverted triangle or uniform load pattern, and thus, the shear wall could fail in shear prior to flexure. This consideration is particularly important when there is an irregularity in the torsion, as in the CW-1977 design (B=2.0) in this paper.

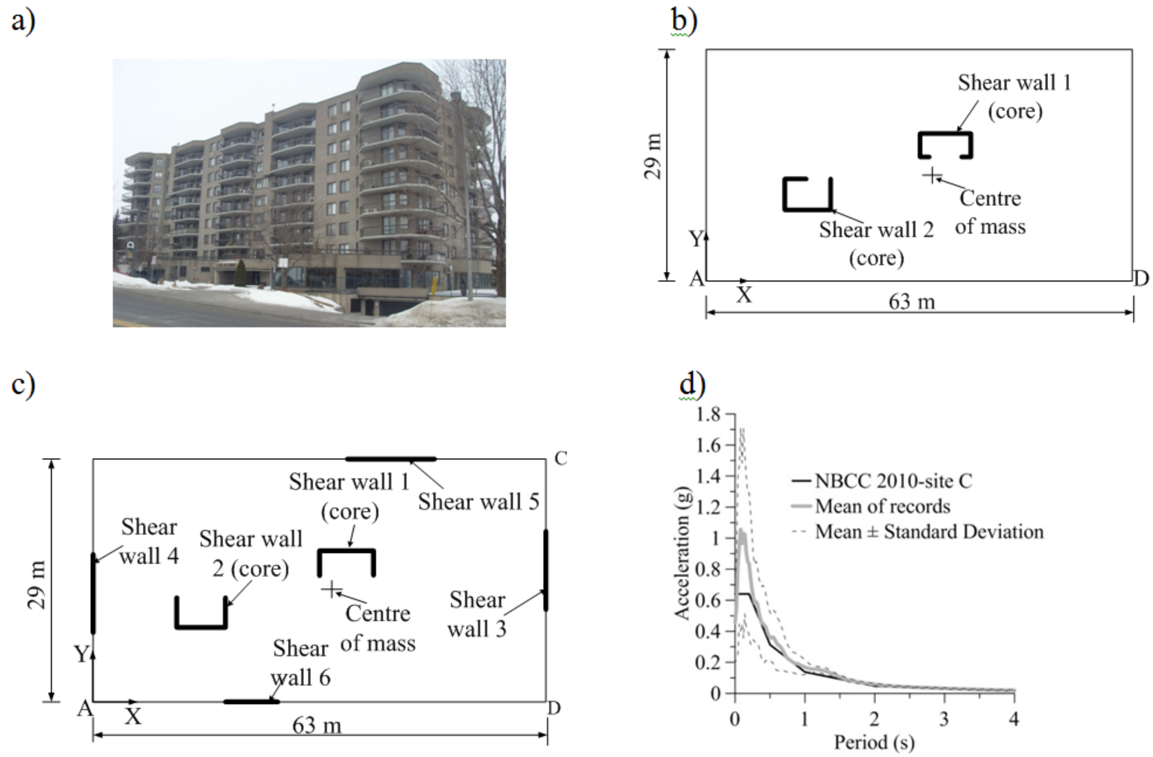


Fig. 1. Building studied: (a) 10-storey RC building; (b) typical plan view; (c) typical plan view with added shear walls; and d) mean response spectrum of the selected ground motion records versus NBCC 2010 design spectrum for site class C.

Therefore, an additional check is added to evaluate whether the response of the individual wall is controlled by shear or flexure. Linear time history dynamic analyses using PERFORM 3D are conducted instead of using a uniform lateral load distribution to classify the walls. These analyses indicate that the two cores designed according to CW-1977 are controlled by shear instead of flexure, as observed when using uniform or triangular load patterns.

In addition, the studies of Ghobanirenani et al. (2012) and Pugh (2012) indicate that shear wall failure is not observed until 1.1 V_n, where V_n is calculated based on ACI 318-11 (2011). Thus, the shear at nominal moment strength is calculated and then compared to 1.1 V_n in this study. The wall is controlled by flexure if the shear at nominal strength is less than 1.1 of the component's shear strength.

After identifying that the inelastic response of the shear wall is controlled by shear or flexure, we shall decide whether the considered action (shear or moment) is controlled by deformation or force. ASCE/SEI 41-13 specifies that moment and shear are normally controlled by deformation. However, the guidelines also prescribe that shear action shall be considered as force-controlled if the shear wall behaviour is controlled by shear and the axial load ratio at the base is equal to or more than 15% (Table 10-20 in ASCE/SEI 41-13). This is the case of the design according to CW-1977 in this study.

ASCE/SEI 41-13 provides different analysis procedures for assessing an existing building. The assessment can be performed using one or more of the following analysis types: i) linear static procedure (LSP), ii) linear dynamic procedure (LDP), iii) nonlinear static procedure (NSP), or iv) nonlinear dynamic procedure (NDP). All of the above analyses were considered in this study.

For linear analysis, the assessment is conducted by comparing demand-to-capacity ratios (DCRs) of the considered action in the component, called "m-factors", for deformation-controlled actions and "J-factors" for force-controlled actions. In this study, the two considered actions for a shear wall are moment and shear. The factor m is dependent on the axial load ratio and the average shear stress and is provided in ASCE/SEI 41-13 for a shear wall controlled by either shear or flexure. The factor J is intended to account for the contribution of additional components (gravity columns) and is dependent on the level of seismicity, the target performance level, and whether actions introduced by adjacent components are expected to remain elastic. In this study, J is equal to 1.0 for the IO performance level and 2.0 for all other performance levels.

For nonlinear analysis, according to ASCE/SEI 41-13, the equivalent plastic hinge rotation (θ_p) over the plastic hinge region at the base of the member is used for a wall with an inelastic response governed by flexure. The acceptable deformation limits for shear walls deforming inelastically under a lateral load and controlled by shear are presented in terms of lateral drift ratios. The drift for multi-storey shear walls is the storey drift.

Component strengths are classified as nominal strength, QCL, for force-controlled actions and expected strength, QCE, for deformation-controlled actions (ASCE 2013). In this study, the expected strength for RC components is equal to 1.25 times the nominal strength.

4. Seismic Assessment of the Studied Building: Results

This section presents the results of the seismic assessment of the building designed according to two alternatives: CW-1977 and MD-2014. The knowledge factor, which accounts for the uncertainty of the build data for the existing building, is set to 0.75 (ASCE 2013). The results for the linear analysis are expressed in terms of the DCRs which are either m for deformation-controlled actions or J for force-controlled actions. In the nonlinear analysis, for deformation-controlled actions to be consistent with linear analysis results, the plastic rotations, θ_p (rotation demand subtracted from yielding rotation determined according to ASCE/SEI 41-13) are normalised and presented by the Γ_θ factor (Eq. (2)), and IO becomes the reference performance level for comparison purposes.

$$\Gamma_\theta = m_{IO} \frac{\theta_d - \theta_y}{\theta_{IO}} \tag{2}$$

where θ_d is the rotational demand from the nonlinear analysis, θ_y is a yielding rotation determined according to ASCE/SEI 41-13, θ_{IO} is the acceptable plastic hinge rotation at the IO performance level, and m_{IO} is acceptable DCR at the IO performance level from linear analysis and is equal to 2.0 in this study (ASCE 2013).

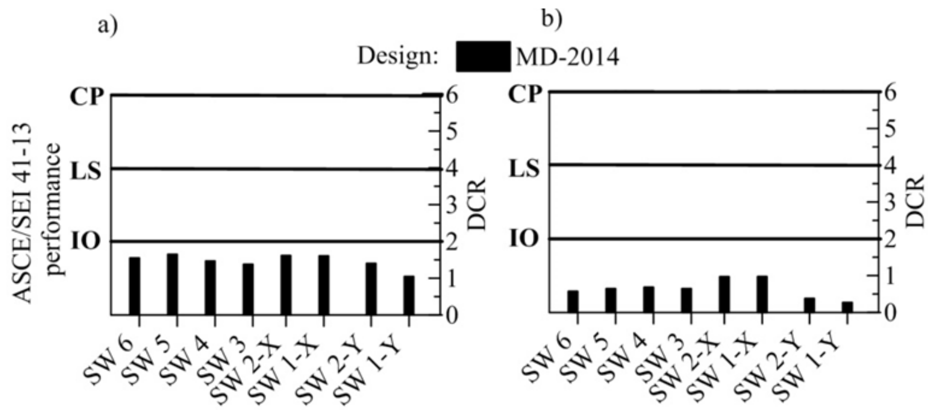
The rotational demand is determined as the equivalent plastic hinge rotations by calculating from the reported load-displacement histories and geometric properties of the walls. The equivalent plastic hinge length of one half the wall lengths is used herein, as prescribed in ASCE/SEI 41-13.

For force-controlled actions, a procedure similar to that used for deformation-controlled actions is applied to the normalised demand drift as follows:

$$\Gamma_\delta = m_{IO} \frac{\delta_d}{\delta_{IO}} \tag{3}$$

4.1. Linear static procedure (LSP, ETABS)

Figure 2 presents the DCRs for both shears and moments from the LSP analyses using ETABS. The analysis is conducted with only the MD-2014 design. The design according to CW-1977 is not considered with this analysis type because the building is highly irregular in torsion (B=2.0) and thus cannot be assessed using static analysis (ASCE 2013).



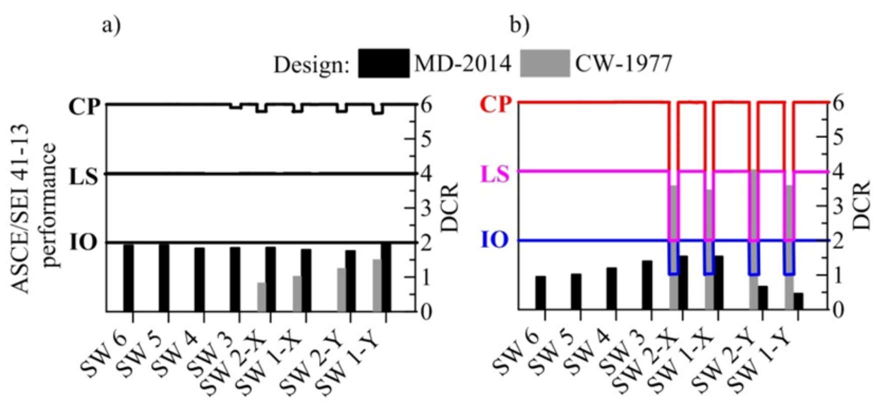
Note: Performance limits depend on axial load ratio and maximum average shear stress in the member, and thus vary with different SW and designs.

Fig. 2. Linear static pushover analysis (LSP) for MD-2014 design: a) moment and b) shear.

The input lateral load is determined using the fundamental period obtained from modal analysis using the ETABS model with effective wall stiffnesses suggested by ASCE/SEI 41-13. The torsional effect is considered by amplifying the force and displacement with the maximum displacement multiplier η of the building, which is equal to 1.26 and 1.05 for the Y and X directions, respectively. The results indicate that the wall performance of the design MD-2014 is in IO limit.

4.2. Linear dynamic procedure (LDP ETABS)

Fig. 3 presents the DCRs for both shear and moment from the LDP using ETABS. The linear dynamic analyses are conducted using modal response spectrum method for a 5% modal damping with the NBCC 2010 design spectra for site class C, as shown in Fig. 1d. The ETABS model is the same as the model used in the LSP. The concurrent multidirectional seismic effect was considered by applying an additional 30% EQ loading perpendicular to the considered direction, as prescribed in ASCE/SEI 41-13.



Note: Performance limits depend on axial load ratio and maximum average shear stress in the member, and thus vary with different SW and designs

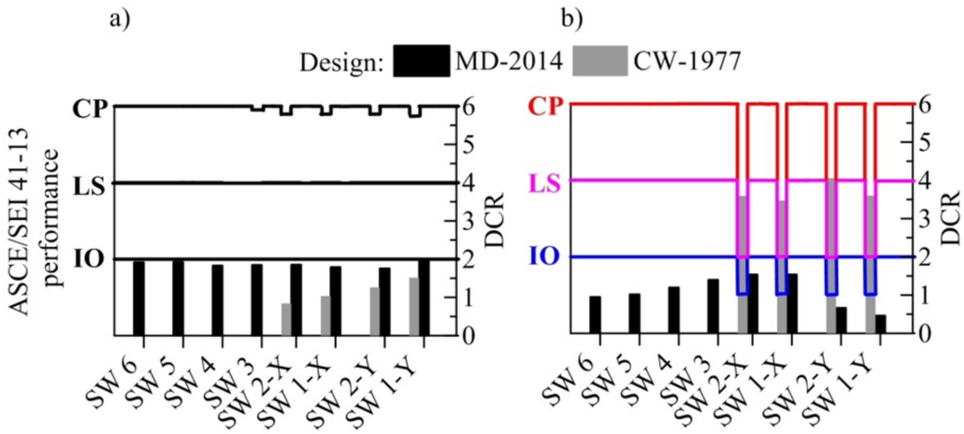
Fig. 3. Linear dynamic analysis (LDP): a) moment and b) shear.

For moments, the results indicate that the design MD-2014 provide wall seismic performance within the IO and LS performance levels. CW-1977 exhibits the best performance with maximum DCR ratio equalling to around 1.75 compared 2.0 in MD-2014. The reason is because there is a significantly larger amount of vertical reinforcement at the base of the walls in the CW-1977 design compared to the MD-2014 design.

Similar to LSP, the seismic performance of the shear response of MD-2014 is within the IO level. However, the walls designed according to CW-1977 are predicted to collapse in shear, and the existing as-built building is seismically unsafe.

4.3. Nonlinear static procedure (NSP PERFORM 3D)

Fig. 4a presents the seismic performance assessment from NSP using PERFORM 3D. The PERFORM 3D model is described in section 4.2. Similar to LSP, the NSP is only conducted with the designs MD-2014.



Note: Performance limits depend on axial load ratio and maximum average shear stress in the member, and thus vary with different SW and designs.

Fig. 4. Nonlinear analyses: a) NSP- static pushover and b) NDP- dynamic.

NSP is conducted using the lateral load pattern from the first mode shape. The ASCE/SEI 41-13 ''Coefficient Method'' was used to determine the Target Drift (TD) ratio, then the TD was amplified by the maximum displacement multiplier η of the building to consider torsion as in LSP. TD ratios along the Y and X directions are 0.37% and 0.31%, respectively. The low TD ratios that have been obtained are due to the ground motions in the ENA region, which results in a low spectral acceleration, Sa, corresponding to fundamental periods (T=1.5 s and 1.4 s for the X and Y directions, respectively) of the building. For the PERFORM 3D NSP, the results for the overall shear wall performances are within the IO performance range.

4.4. Nonlinear dynamic procedure (NDP PERFORM 3D)

The final assessment is the NDP using PERFORM 3D. The modelling parameters are the same as for the NSP. The damping is represented by a Rayleigh model, and the assumed damping ratio is 2.0%, assigned in modes 1 and 2. A set of 12 ground motions corresponding to the building site is used; consistent with those used for the previous studies (Luu et al. 2014). The 12 ground motions were selected and scaled according to the recommendations from Atkinson (Atkinson 2009). The mean acceleration response spectrum of the scaled ground motions is presented in Fig. 1d. The mean spectrum is in good agreement with the design spectrum prescribed in NBCC 2010 used for the LSP, LDP, and NSP (Fig. 1d). The mean results for a set of 12 ground motions are used to evaluate the seismic performance of the three alternative designs.

For the walls designed according to CW-1977, which have inelastic responses controlled by shear, the factor Γ_δ presented in Fig. 4b is the product of δ_d/δ_{IO} with m_{IO}. This adjustment is made to obtain coherent comparisons between walls that have inelastic response controlled by flexure and shear in Fig. 3b. The concurrent multidirectional seismic effects are considered by a similar method as in the LDP.

The results of the NDP indicate that the buildings designed according to MD-2014 are within the LS performance level. However, the CW-1977 design, which contains only two cores, exhibits unsafe performance. Both cores are predicted to collapse by shear failure.

5. Comparisons of different assessment procedures and recommendations

Comparisons of the seismic DCRs (Γ_θ, Γ_δ or m) obtained from static and dynamic procedures are shown in Table 2. The static procedures (LSP and NSP) produced smaller seismic DCRs than the DCRs computed from the dynamic procedures (LDP and NDP) (Table 2). This result is not consistent with some previous studies (Hagen 2012; Gonzales and López-Almansa 2012), in which the static procedure was more conservative than the dynamic procedure. In this study, the static procedure predicts lower DCRs than dynamic procedure because of the high-predominant-frequency ground motions (on the order of 10 Hz), which are typical in the ENA region of this building. This implies that the HMEs for walls located in ENA are more important than the consideration in ASCE/SEI 41-13. In this building, in the Y direction of the NBCC 2010 design, the contribution to the base shear force demand from the LDP of the second mode is 1.13 times the LDP of the first mode. Thus, the static procedure (LSP or NSP) should be used with caution for buildings in which HMEs are expected to be important. We therefore recommend the use of linear dynamic analysis (LDP or NDP) for shear wall buildings located in ENA.

The comparisons between the Γ_θ or Γ_δ results obtained from linear (LSP and LDP) and nonlinear (NSP and NDP) analyses are shown in Table 3. The seismic performance level is varying significantly with the analysis procedure used. The variation is more considerable for static analyses than dynamic analyses. SW3 shows IO performance level with LDP but LS with NDP.

Table 2. Comparisons of seismic DCRs (Γ_θ , Γ_δ or m) between static and dynamic procedures

Design		SW	1	2	3	4	5	6
MD-2014	LI[1]	X	1.6[1]/1.8[2]	1.6/1.9	-	-	1.7/1.9	1.6/1.9-
		Y	1.0/2.0	1.4/1.8	1.4/1.9	1.5/1.8	-	
	NL[2]	X	1.2/1.8	1.3/1.9	-	-	1.3/1.4	1.3/1.5
		Y	1.7/2.7	1.7/1.9	1.7/3.0	1.8/2.3	-	-

Note: LI=linear; NL= nonlinear; the ratio between Γ_θ, Γ_δ or m obtained from static[1] and dynamic[2] analyses; for linear analyses, the maximum m value between shear and moment actions is selected.

Table 3. Comparisons of seismic DCRs between the linear and nonlinear procedures

Design		SW	1	2	3	4	5	6
MD-2014	ST	X	1.6[1]/1.2[2]	1.6/1.3	-	-	1.7/1.3	1.6/1.3
		Y	1.0/1.7	1.4/1.7	1.4/1.7	1.5/1.8	-	-
	DY	X	1.8/1.8	1.9/1.9	-	-	1.9/1.4	1.9/1.5
		Y	2.0/2.7	1.8/1.9	1.9/3.0	1.8/2.3	-	-
CW-1977	DY	X	3.5/3.0	3.6/2.9	-	-	-	-
		Y	3.6/3.4	4.0/3.8	-	-	-	-

Note: ST= static; DY= dynamic; the ratio between Γ_θ, Γ_δ and m obtained from linear[1] and nonlinear[2] analyses; for linear analysis, the maximum m value between shear and moment actions is selected.

6. Summary and conclusions

This paper presented seismic assessment of two alternative design approaches for RC shear walls subjected to high frequency content ENA ground motions. The first design approach (1) is without considering shear force and moment amplification due to nonlinear higher mode effects (HMEs) (CW-1977). The other shear wall design methodology (2) explicitly considers HMEs using shear, ϖ_v from A23.3-2014 (21.5.2.2.7) and moment amplification factors with suitable envelopes along the wall height (MD-2014). An existing building designed according to CW-1977 was selected and redesigned according to MD-2014. The seismic performance of the shear walls designed according to these two design approaches was assessed. The assessment followed the acceptance criteria prescribed in ASEC/SEI 41-13 (ASCE 2013). All procedures prescribed in ASCE/SEI 41-13, including the LSP, LDP, NSP, and NDP, were used to assess the building. The study shows a potential use of ASCE/SEI 41-13 to assess shear wall buildings located in eastern North America. Using the acceptance criteria of ASCE/SEI 41-13, the design according to MD-2014, which considers nonlinear HMEs, achieve both the IO and LS performance levels. The design according to CW-1977 that was highly irregular in torsion and which does not take into account HMEs, is unsafe because of shear failure. The results indicated that static procedures provided lower demand capacity ratios than dynamic procedures because of the significant HMEs in the ENA region.

The obtained results in this study are based on damage threshold values of shear walls located in the west. Additional studies are needed to determine appropriate acceptance criteria for walls located in ENA.

7. Acknowledgements

The authors thank the National Science and Engineering Research Council of Canada (NSERC) and the Quebec Fund for Research in Nature and Technology (FQRNT) for providing the financial support.

8. References

ACI. “Building Code Requirements for Structural Concrete and Commentary”. *American Concrete Institute*. ACI 318-11 .Farmington Hills, Mississippi. USA. 2011.

ASCE 41-13. “Seismic Rehabilitation of Existing Buildings. ASCE/SEI 41-06”, *American Society of Civil Engineers*, Reston, VA, 2013.

Atkinson, G.M. “Earthquake Time Histories Compatible with the 2005 NBCC Uniform Hazard Spectrum”. *Can. J. of Civ. Eng*., 36(6), 2009, pp. 991-1000.

Birely, A. C., Lowes, L. N., and Lehman, D. E. 2014. “Evaluation of ASCE 41 Modeling Parameters for Slender Reinforced Concrete Structural Walls”. *American Concrete Institute*, Special Publication. 297 (1-18). ISBN: 9780870318675

CSI. PERFORM-3D. “Nonlinear Analysis and Performance Assessment for 3D Structures”, V.5.0.1. *Computers and Structures*, Inc., Berkeley, CA, USA, 2012.

CSI. “ETABS: Structural Analysis Program”,V 9.7.2 *Computer and Structures*, Inc., Berkeley, CA, USA. 2010.

CSA. “Design of concrete structures”. CSA standard A23.3-14, *Canadian Standards Association*, Toronto, Ont. Canada. 2014.

CSA. “Design of concrete structures”. CSA standard A23.3-04. *Canadian Standards Association*, Toronto, Ont. Canada. 1973.

Dezhdar, E. and Adebar, P. “Estimating Seismic Demands on High-Rise Concrete Shear Wall Buildings”, *15th World Conference on Earthquake Engineering*, Lisbon. 2012, 10pp.

Hagen, G.R. “Performance Based Analysis of a Reinforced Concrete Shear Wall Building”, Master thesis. Faculty of *California Polytechnic State University*, San Luis Obispo, California, 2012, USA.

Ghorbanirenani, I., Tremblay, R., Léger P., and Leclerc M. “Shake Table Testing of Slender RC Shear Walls Subjected to Eastern North America Seismic Ground Motions”. *ASCE J. Struct. Eng.*, 138(12), 2012, pp. 1515-1529.

Gonzales, H. and López-Almansa, F. “Seismic Performance of Buildings with Thin RC Bearing Walls”. *Eng. Struct.,* 34, 2012 pp. 244–258.

Luu, H. “Earthquake response analysis and resistant design of moderately ductile reinforced concrete shear walls considering higher mode effects”. Ph.D. Thesis, *École Polytechnique de Montréal*, Departement des génies civil, géologique et des mines, Montréal, Qué, Canada. 2014.

Luu, H., Léger, P. and Tremblay, R. “Seismic Demand of Moderately Ductile Reinforced Concrete Shear Walls Subjected to High-Frequency Ground Motions”. *Can. J. Civ. Eng.*, 41(2), 2014b, pp. 125-135.

NRCC. “National Building Code of Canada; Part 4: Structural Design”. Canadian Commission on Building and Fire Codes, *National Research Council of Canada (NRCC)*, Ottawa, Ont, Canada. 2010.

NRCC. “National Building Code of Canada; Part 4: Structural Design”. Canadian Commission on Building and Fire Codes, *National Research Council of Canada (NRCC),* Ottawa, Ont, Canada. 1977.

Pugh J. “Numerical Simulation of Walls and Capacity Design Recommendations for Walled Buildings”. PhD Thesis, *University of Washington*, USA, 2012.