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A Simple Method for Determining Seismic Demands on Gravity Load Frames

J. Beauchamp, P. Paultre, and P. Léger

Abstract: This paper presents a simple method based on modal response spectrum analysis to compute internal forces in structural elements belonging to gravity framing not part of the seismic force resisting system (SFRS). It is required that demands on these gravity load resisting system (GLRS) be determined according to the design displacement profile of the SFRS. The proposed new method uses the fact that if the linear stiffness properties of the GLRS not part of the SFRS have negligible values compared to those of the SFRS, only the latter will provide lateral resistance. Displacements of the GLRS then correspond to those of the SFRS alone. The new method is illustrated by computing the seismic responses of a symmetric and an asymmetric 12 storeys reinforced concrete (RC) building. These results are compared to those obtained from the application of the simplified analysis method proposed in the Canadian standard for the design of concrete structures. Non linear time history analyses are also performed to provide a benchmark for comparison. Results show that the new method can predict shear and bending moment in all members at once with ease. Therefore, this new simplified method can effectively be used to predict seismic forces in elements not considered part of the SFRS.

Key words: Reinforced concrete, gravity frames, shear wall, coupled walls, simplified analysis method, non linear time history analysis, CSA A23.3-14.

Résumé : Cet article présente une nouvelle méthode pour calculer les efforts internes dans les éléments du système de résistance aux charges gravitaires (SRCG) ne faisant pas partie du système de reprise des forces sismiques (SRFS) basée sur l'analyse dynamique linéaire. Les codes canadien et américain exigent que ces efforts soient déterminés en fonction de la configuration de déplacement correspondant au tremblement de terre de dimensionnement. Cette nouvelle méthode est basée sur le fait que si les propriétés de rigidité élastique des éléments ne faisant pas partie du SRFS sont négligeables comparées à celles du SRFS, seulement ce dernier participe à la résistance latérale et les déplacements du SRCG correspondent à ceux du SRFS seul. Le bien-fondé de la nouvelle méthode est démontré par le calcul de la réponse sismique de bâtiments de 12 étages en béton armé symétrique et asymétrique. Ces résultats sont comparés à ceux obtenus à partir de l'application de la méthode d'analyse simplifiée proposée dans la norme de calcul des structures en béton (CSA

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A23.3-14, art. 21.11.2.2.). Des analyses non linéaires temporelles sont également effectuées afin de fournir une base de comparaison. Les résultats montrent que la nouvelle méthode peut facilement prédire l'effort tranchant et le moment de flexion dans toutes les membrures à la fois. Par conséquent, cette nouvelle méthode d'analyse peut être effectivement utilisée pour prédire les forces sismiques dans le SRCG.

Mots clés : Béton armé, ossature résistant aux forces de gravité, mur de cisaillement, murs couplés, méthode d'analyse simplifiée, analyse non linéaire temporelle, CSA A23.3-14.

1. Introduction

In Canada (National Research Council Canada 2010) and the United-States (American Society of Civil Engineers 2013), the seismic design process starts with the selection of the seismic force resisting system (SFRS) that is required to entirely resist the design seismic forces. This part of the design process is quite straightforward as it simply requires to analyse a finite element model that does not include elements not part of the SFRS. For the rest of the structure, the gravity load resisting system (GLRS) made of members not designated as part of the seismic force resisting system, two design approaches are available. They can be designed to either possess (1) sufficient capacity to deform elastically or (2) sufficient ductility to sustain lateral displacements induced by the SFRS (CSA 2014; ACI committee 318 2014). The Canadian design of concrete structures standard and the American Concrete Institute code (CSA 2014; ACI committee 318 2014) give detailing requirements depending on the induced force demands by the imposed lateral displacement of the SFRS.

The elastic capacity approach is particular, because it requires an analysis of the complete structure to determine seismic internal forces in elements part of the GLRS in the deformed configuration. The difficulty is that, while considering that all the seismic demand is taken by the SFRS results in a safe design of the latter, it leaves no clue as to how much seismic forces is carried in the rest of the structure. It is then impossible to rationally allot part of the seismic forces to the elements that are not part of the SFRS. For this reason, the default method specified in the NBCC for the GLRS is a non linear time history (NLTHA) analysis accounting for inelastic deformations in the SFRS. This gives a realistic estimate of how the entire building would really behave in an earthquake and thus allows for the determination of design forces in the GLRS. However, NLTHA are tedious, requires specialized skills and are not typically needed for common buildings, thus making it impractical for the analysis of the GLRS of simple common buildings. Alternative simplified methods are required to achieve an equally safe design more easily. One such simplified method is proposed as an alternative in the new edition of the standard for the design of concrete structures (CSA 2014). This displacement-based method is detailed in section 2 of this paper. A new alternative method (section 4) that assign loads to the GLRS without reducing those on the SFRS is proposed herein. Its main advantages are its simple implementation with commonly used finite element models and conservative results as compared to the reference NLTHA results. In addition, the method uses only finite elements models required by NBCC, i.e., a full model of the structure and a model of SFRS. Indeed, the method can obtain all required force demands from the analysis of a single full model of a building.

Determination of force demand on GLRS elements is important because failure of these elements is one of the main causes leading to the collapse of reinforced concrete (RC) buildings during earthquakes. For example, during the 1994 Northridge earthquake (Mitchell et al. 1995), some frames designed only for gravity loads failed and caused the ruin of the buildings. This was also the case for damage to some engineered structures during the 2010 Haiti earthquake (Boulanger, Lamarche, Proulx and Paultre 2013) and the total collapse of some buildings during the 2011 Christchurch earthquake (Elwood 2013). An accurate and simple approach is needed to properly account for seismic forces induced in these elements. As mentioned, NLTHA are tedious and the current simplified method in CSA A23.3-14 would benefit from further applications and comparisons with other types of 3D buildings, such as coupled walls and frame SFRS, particularly if the torsional response is significant.

This paper presents and apply a simplified method that can, with as much accuracy as that of the alternative method in the CSA A23.3-14, predict internal forces in GLRS elements while being easier to implement in a design context as all force demands are directly obtained for all members without any additional calculations. The paper also clarifies some hypotheses behind alternative analysis methods for the GLRS seismic design. First, the current A23.3-14 alternative simplified method is reviewed. Modelling assumptions related to the proposed method are then presented with applications to two 12-storey reinforced concrete (RC) flat slab shear wall buildings.

The scope of this paper is limited to predicting translational and rotational displacement demands in reinforced concrete columns even though the proposed method can also be used to predict displacement demand in beams in frame-wall structures. From the displacement demand, forces in the GLRS's elements can be obtained. The seismic contribution to axial forces in columns was investigated, but it was found insignificant when combined with gravity axial loads. Therefore, it is not presented in this paper. In all analyses, shear deformation of the shear walls and foundation rotation are not considered to simplify calculations, to focus on interactions between the SFRS and the GLRS and to facilitate comparisons among the different analysis methods. These aspects can, however, be included in the analysis procedure presented in this paper.

2. Current simplified method in CSA A23.3-14

The 2014 edition of the Canadian standard Design of Concrete Structures CSA A23.3-14 specifies that the GLRS has to be laterally displaced according to the design displacement profile. Because the total seismically induced forces must be resisted by the SFRS, the design displacements are computed accordingly. Also, the analysis shall account for the SFRS' inelastic displacement profile using a non linear model of the SFRS or a linear model with appropriately reduced stiffness.

If the SFRS is composed of shear walls or coupled walls, a simplified method is recommended to obtain force demands in GLRS elements. Figure 1 presents the envelope of drift ratio that can be used to compute locally force demand on critical columns or globally displacement profile representative of the SFRS inelastic deformations. These displacements can then be applied to a linear model of the building for a fast evaluation of GLRS element forces. However, considering that one of the advantages of the finite element method (FEM) is to give design forces and displacements in a single computation cycle, it can conveniently be used to obtain all required force demands using the proposed method. When torsion is considered (as it should be), the top displacement is different for each column. Therefore, multiple profiles must be computed and applied to the model, thus requiring more computing time when compared to the proposed method (see sections 3 and 4) which can give the forces for all members of the GLRS with a single load case per seismic loading direction.

The simplified drift ratio profile presented in the CSA A23.3 standard was obtained from multiple non linear analyses performed on thirteen cantilever shear walls with fixed support and with a rotational spring at the base whose stiffness was varied to model different rotation magnitude at the base (Dezhdar 2012; Dezhdar and Adebar 2015). Dezhdar and Adebar (2015) defined the drift ratio value of 0.7 at the base of a structure to account for increase inter-story drift at the base of a shear wall due to deformation of foundation, inelastic curvature in storeys that extend below the base and also shear strains.

3. Accounting for core wall inelastic deformation in linear analysis

When computing seismic forces and displacements in core wall buildings from response spectrum analysis, it is required by current standards to reduce the gross stiffness properties of structural elements to account for concrete cracking and reinforcement yielding. A well-known equation, proposed by Paulay (1986), to determine the effective inertia, I_e , over the full height of a shear wall is

$$[1] \quad I_e = \left(0.6 + \frac{P}{f'_c A_g} \right) I_g$$

Fig. 1. Envelope of relative inter-story drift ratio for simplified analysis of core-wall buildings in CSA A23.3-14

where P is the axial load acting on the wall, f'_c is the concrete compression strength, A_g is the gross concrete area of the wall and I_g is the gross concrete section moment of inertia. With a typical axial load for a wall $P = 0.10f'_cA_g$, this equation gives $I_e = 0.70I_g$. Equation [1] was included in the 2004 edition of the CSA A23.3 Standard Design of concrete structures. In the CSA A23.3-14 Standard, the following equation proposed by Adebar and Dezhdar (2015) to determine the effective stiffness of walls was adopted:

$$[2] \quad I_e = \left[1.0 - 0.35 \left(\frac{R_d R_o}{\gamma_w} - 1 \right) \right] I_g, \quad 0.5I_g \leq I_e \leq 1.0I_g$$

where γ_w is the wall overstrength factor equal to the ratio of the load corresponding to nominal moment resistance of the wall system to the factored load on the wall system, R_d and R_o are the ductility-related and overstrength-related force modification factors, respectively. For the core wall building shown in Fig. 2, equation [2] gives, for the coupled and cantilever directions respectively, $I_e = 0.62I_g$ and $I_e = 0.5I_g$.

Such equations give an estimate of the structure's global behaviour, but does not reflect the concentration of deformations that occurs at plastic hinge locations. This behaviour is better captured through non linear analysis, but research has shown that a linear model can be modified to accurately represent seismic forces and deformations in the plastic hinge zone. For example, Dezhdar (2012) estimated the curvature demand of shear walls by running response spectrum analyses on models with reduced stiffness at the base and mid-height. A23.3-14 Standard also hints toward this approach by stating that "in lieu of using a non linear model of the SFRS, a linear model with appropriately reduced properties at plastic hinge locations may be used to estimate the inelastic displacement profile" (CSA A23.3-14, art. 21.11.2.1 b). This led to the development of the analysis procedure corresponding to the results identified latter on as linear plastic hinge (LPH). The idea with this procedure is to reduce the core wall elastic modulus with a global coefficient as described in equation [2] and to reduce it furthermore in the

1 expected plastic hinge location. That way, deformations are more important in the plastic hinge zone.
 2 Many equations have been developed to estimate the plastic hinge length. It is common to express them
 3 with the following parametric formula:

$$4 \quad [3] \quad \ell_p = \alpha \ell_w + \beta h_w + \gamma f_y d_b$$

5 where α , β and γ are constant parameters, ℓ_w is the wall length, h_w is the wall height, f_y is the
 6 yield stress of the reinforcement and d_b is the diameter of the reinforcement bars. In the CSA A23.3
 7 Standard, α varies between 0.5 when calculating the rotation capacity of walls and 1.0 when calculating
 8 the rotation demand on walls with $\beta = 0$ and $\gamma = 0$. Using these coefficients, the plastic hinge
 9 length would vary between a minimum of 3.1 m to a maximum of 8.4 m. According to CSA A23.3-14
 10 Standard, special reinforcement must be provided over a minimum height of plastic hinge region equal
 11 to :

$$12 \quad [4] \quad h_p = 0.5 \ell_w + 0.1 h_w$$

13 In the single wall direction, $h_p = 0.5 \times 6.4 + 0.1 \times 48.65 = 8.1$ m. In the coupled direction,
 14 $h_p = 0.5 \times 8.4 + 0.1 \times 48.65 = 9.1$ m. Results presented in section 6.4 have shown that plastic hinge
 15 region reduced elastic modulus equal to half that of the rest of the core wall gives satisfactory results
 16 when comparing linear analysis with effective stiffness and rigorous nonlinear analyses. A wall average
 17 stiffness reduction of $E_{\text{ceff}} = 0.7E_c$ has been used in both directions. Accounting for cracking and for
 18 tension shift, a reduced average stiffness $E_{\text{ceff}} = 0.35E_c$ has been used over an average plastic hinge
 19 region height of 8.5 m corresponding to the height of the first two storeys. This value was determine by
 20 trial and error as explained later. Dezhidar (2012) has related the stiffness reduction factor in the plastic
 21 hinge length equal to $0.5\ell_w$ to the force reduction factor equal to the ratio of the factored overturning
 22 moment, M_f , over the nominal bending moment resistance, M_n , at the base. For a force reduction
 23 factor equal to or larger than 2.5, it is recommended to use a stiffness reduction of approximately 0.18.
 24 The stiffness reduction factor increases linearly from 0.18 to 0.5 at a force reduction factor equal to 1. In
 25 the case of the building studied, the force modification factors are 2.75 in the single wall direction and
 26 2.2 in the coupled direction, for an average force reduction factor of about 2.5. Note that the stiffness
 27 reduction factor used in this research is about twice the value recommended by Dezhidar (2012) but
 28 applied on about twice the length of wall.

29 4. Proposed simplified modal response spectrum method

30 The proposed method is based on modal response spectrum analyses (RSA) to avoid using the more
 31 complicated non linear time history analysis methods. The proposed method uses two 3D structural
 32 models, one representing only the SFRS and the other representing the complete building. As required
 33 by NBCC 2010, these two models are used to determine the building's fundamental period to calculate
 34 the lateral seismic force and to design the SFRS. Once the information needed from the complete
 35 building model is extracted, the axial, shear and bending stiffnesses of its GLRS elements are reduced
 36 to a small fraction of their initial stiffness values by multiplying them by a factor F_{sr} equal to 10^{-2}
 37 to 10^{-3} . This factor is determined so that the complete building model with reduced GLRS stiffness
 38 has quasi similar modal characteristics as the SFRS alone. In this study, it was found that $F_{sr} = 10^{-2}$
 39 gave good results. This method is labelled GLRS with nearly null stiffness (GNS). Response spectrum
 40 analysis of this model produce nearly vanishing GLRS element internal forces corresponding to those
 41 that occur when all seismic loads is attributed to the SFRS. The design forces F_{GLRS} in these elements
 42 are determined as follows:

$$43 \quad [5] \quad F_{GLRS} = F_{GNS} \times \frac{1}{F_{sr}} \times \frac{V_d}{V_e} \times \frac{R_d R_o}{I_e}$$

where V_d is the lateral earthquake design force at the base of the structure, V_e is the lateral earthquake elastic force at the base of the structure, R_d is the ductility related force modification factor, R_o is the overstrength-related force modification factor, I_e is the earthquake importance factor as defined in the NBCC (National Research Council Canada 2010) and F_{GNS} are the reduced forces acting in the elements from the GNS.

The implementation of the new simplified methods is done as follows :

1. Prepare model 1 of the SRFS.
2. Prepare model 2 of the complete structure (SRSF and GLRS). Note that model 1 can be obtained from model 2 by reducing the stiffness of the GLRS elements as indicated in step 4.
3. Compute V_d , V_e in accordance with the NBCC.
4. In model 2, reduce the stiffness of the GLRS by multiplying by F_{sr} to get the GNS model.
5. Using a copy of the GNS model, reduce the stiffness of the plastic hinge zone to get the LPH model (section 3).
6. Perform RSA analyses with GNS model and LPH models.
7. Compute internal forces in the elements of the GLRS from equation [5] for GNS and LPH stiffness reduction.

In contrast, if the envelope of drift ratio simplified method of the CSA A23.3-14 (CSA) method is used, the top column displacements obtained from analysis of the GNS model are used as input values to compute the drift ratios and impose lateral displacements profiles for each column.

5. Application of the proposed method

For each building studied in this project, three different linear analysis methods and one non linear time history analysis are used. The three linear analyses using (i) the simplified the envelope of drift ratio method of the CSA A23.3-14, (ii) the GLRS with nearly null stiffness (GNS), and (iii) the linear plastic hinge with reduced stiffness (LPH) are carried out using RSA with ETABS (Computer and Structures, Inc. 2010) finite element program. Non linear time history analyses (NLTHA) are carried out using SeismoStruct software (Seismosoft 2013b) to produce reference values.

5.1. Studied buildings

Two sample buildings are analysed to study the proposed methods. The structure shown in Figs. 2 and 3 is taken from the Cement Association of Canada's Concrete design handbook (Mitchell and Paultre 2006). This 12 story RC building has a central core wall which is a cantilever wall in the north-south direction and a coupled wall systems in the east-west direction. The studied buildings are located in Montreal on stiff soil, which is a site class D according to the NBCC-10. Table 1 presents the design spectrum from NBCC 2010 for Montreal for soil site class D, using site modification factors $F_a = 1.144$ and $F_v = 1.360$. The SFRS consists of coupled ductile walls and ductile cantilever walls for the E-W and N-S directions, respectively. The force reduction factors related to the ductility and overstrength are, as defined by the A23.3-04 Standard, $R_d = 4.0$ and $R_o = 1.7$ for the coupled walls and $R_d = 3.5$ and $R_o = 1.6$ for the cantilever walls.

The building shown in Figs. 4 and 5 is identical to the symmetrical building, but with the core offsets 6 meters to the north. Both buildings are then very similar and they allow for an investigation of torsional effects. Torsion sensitivity is assessed from NBCC (NBCC, art. 4.1.8.11.9) by evaluating the parameter B_x defined as follows:

$$[6] \quad B_x = \delta_{max} / \delta_{ave}$$

Table 1. Spectral response accelerations and design spectral response acceleration

Priode T , s	0	0.2	0.5	1.0	2.0	≥ 4.0
S_a , g	0.64	0.64	0.31	0.14	0.048	0.024
S , g	0.732	0.732	0.422	0.190	0.065	0.033

Table 2. Element properties reduction factors for linear analysis

Element type	Effective property
Column, stories 1-3	$I_e = 0.65I_g$
Column, stories 4-9	$I_e = 0.60I_g$
Column, stories 10-12	$I_e = 0.55I_g$
Coupling beam	$A_{ve} = 0.45A_g$; $I_e = 0.4I_g$
Slab frame element	$I_e = 0.2I_g$
Wall	$E_{ceff} = 0.7E_c$

where δ_{max} is the maximum displacement and δ_{ave} is the average displacement of the structure at level x . The maximum value of B_x is $B = 1.77$ for the symmetric building and $B = 4.00$ for the offset core building. For the symmetric building, B_x is relatively constant, but there is a concentration of torsional displacements near the base of the offset core building. Because of the relatively large B factors, torsional effects must be evaluated through RSA or non linear dynamic analysis (NBCC, art. 4.1.8.11.10b).

5.2. Sections and detailing

The design of the buildings was carried out according to CSA A23.3-04 and NBCC 2010. Because the offset core building shown in Fig. 4 has the same SFRS as the symmetric building and because minimum reinforcement requirements mostly govern the design, the same detailing is used. Vertical reinforcement patterns for the core wall are shown in figure 6. It consists in 4-25M of concentrated reinforcement at each end and corner of the "C" wall and 10M at 200 mm c/c of distributed reinforcement elsewhere. Table 2 shows the CSA A23.3-14 stiffness reduction factors used for structural elements prior to the additional reduction factors presented in section 3 for the linear plastic hinge (LPH) method. The CSA A23.3 Standard recommends to use upper bound values for the stiffness of the GLRS elements so as not to underestimate the demand place on them. The values used herein correspond to the recommended values in the A23.3 Standard but were checked to make sure that they were reasonable. Upper bound values 25 % larger than what is indicated in Table 2 was judged reasonable. Because the stiffness reduction factor is constant for all members, the forces demands placed on the columns would be simply 25 % larger than the values presented in the figures. To avoid overcrowded figures, these upper bound force distributions are not shown in the figures. Of course, new analyses would have to be carried out if the upper bound stiffness reduction factor is not constant for all members. One approach would be to develop moment-curvature response of the different elements in the GLRS and use secant values of stiffness at the appropriate load levels as input values for the analyses. This is one important advantage of the method where parametric studies can be performed to assess the influence of different stiffness assumptions.

Fig. 2. Plan and elevation of the symmetric building

Fig. 3. Three dimensional view of the symmetric building

5.3. Finite element modelling strategy

To accurately represent the buildings, the following modelling strategy is intended to be as simple and straightforward as possible. For the linear model using ETABS, elastic frame elements are used to model the slabs and columns while the walls are modelled with elastic shells. For the NLTHA models using SeismoStruct, the NBCC’s requirement is simply that the wall’s inelastic profile be taken into account. Moreover, because of the hypothesis that all inelastic deformations are concentrated in the SFRS, only the latest is modelled with non linear elements. The rest of the structure, the columns and the slabs, are modelled with elastic frame elements, just as in the linear models. The walls are discretized with inelastic rectangular fibre elements with the flange walls linked to the web wall through rigid link constraints. Figure 6 shows details of the wall reinforcement. In the fiber elements model of the C-shape walls, half of the corner area is assigned to the web wall and the other half is assigned to the flange walls. According to Beyer et al. (2008), this is the preferred discretization to model U-walls with inelastic wide-column. Fibre based sections are used to obtain the sectional stress-strain state of the element through integration of the materials’ uniaxial non linear constitutive laws.

5.3.1. Shear deformation in walls

Kara and Dundar (2009) stated that the influence of shear deformation increases with lateral loads and as the aspect ratio of shear walls decreases. A parametric study by (Huang and Kwon 2015) shows that for period larger than $T = 1.0$ s, accurately capturing the entire hysteretic shear behaviour does not greatly improve the accuracy of the global displacement results and fibre section models may be used. Additionally, if the structure is not shear critical, meaning it is flexure critical, fibre section models can be used regardless of period. The criteria proposed by Huang and Kwon (2015) to determine the failure

Fig. 4. Plan and elevation of the offset core building

Fig. 5. Three dimensional view of the offset core building

mode is called the shear force demand-capacity ratio and is expressed as:

$$[7] \quad I_v = \frac{M_r}{V_r h_w}$$

where M_r is the bending moment resistance and V_r is the shear force resistance at the base of a wall with height h_w . If I_v is larger than 1, the element is shear critical and if it is less than 1, the element is expected to exhibit a flexural failure mode.

For the moment M_r and shear V_r resistances at the base and the length h_w of the considered walls, I_v varies from 0.085 to 0.229, meaning the SFRS is flexure critical. According to the work of Huang and Kwon (2015), shear deformations does not need to be explicitly modelled. However, Bazargani (2014) claims that, even though shear deformation is negligible compared to the global lateral displacement of a RC building, it can be a significant part in the plastic hinge region. Nonetheless, it was decided to neglect the hysteretic shear behaviour of the walls for simplicity while comparing different analysis methods.

5.3.2. Material constitutive laws

The modelled buildings are assumed to be made entirely of concrete with 30 MPa compressive strength. The elastic modulus $E_c = 24974$ MPa and the Poisson's ratio $\nu = 0.2$ are used to define the material used for all linear frame members. For modelling the nonlinear fiber-element wall members, the concrete material law is defined with an uniaxial constant confinement model based on the constitutive relationship proposed by Mander et al. (1988) and the reinforcing steel follows the Menegotto-Pinto steel material law (Menegotto and Pinto 1973). As implemented by Monti et al. (1996), this

Fig. 6. Details of wall reinforcement

1 model includes the hardening rules proposed by Filippou et al. (1983). According to SeismoStruct's
 2 user manual Seismosoft (2013a), this material curve is specifically designed for reinforcement in RC
 3 structures. Parameters are calibrated to represent $f_y = 400$ MPa steel, with typical values for the elastic
 4 modulus $E_s = 200$ GPa and the strain hardening ratio $\mu = 0.005$.

5.4. Consideration of torsion

6 Various recommendations are made in Canadian code and standard to take into account torsion
 7 in seismic analysis. The NBCC 2010 (National Research Council Canada 2010) recommends to use
 8 a static torsional moment with an accidental eccentricity equal to $\pm 0.10D_{nx}$. Three-dimensional dy-
 9 namic analysis with offset centre of mass is allowed for non torsionally sensitive buildings ($B < 1.7$).
 10 However, other codes, such as EC-8:2004 and IBC 2012, recommend a 5% value. Pekau and Gui-
 11 mond (1990), among others, also recommend a 10% eccentricity value as they found that the 5% value
 12 does not include non simultaneous degradation effects. Also, other researchers suggest that accidental
 13 eccentricity may often be lower than 5% (Ramadan et al. 2008; Anagnostopoulos et al. 2015).

14 To avoid overestimating accidental torsion and to get comparable results from each analysis pro-
 15 cedure, it is chosen to offset nodal masses by $\pm 0.05D_{nx}$ in each direction for linear and non linear
 16 analyses. A combination is performed for two orthogonal directions. For RSA analyses, this is done by
 17 adding 100 % of the seismic load effect in one direction with 30 % of that of the perpendicular direc-
 18 tion. For NLTHA, two perpendicular components of the ground motion recordings are simultaneously
 19 applied.

5.5. Ground motion records selection

Ground motions acceleration records are selected to use in NLTHA. Nine artificial accelerograms are selected from Atkinson’s database (Atkinson 2015) and seven historical records from Pacific Earthquake Engineering Research Center’s NGA West2 database (Pacific Earthquake Engineering Research Center 2014). The adopted selection method consists of choosing the records for which the spectral shape best matches the design spectrum for a specified period range. The selected accelerograms are then linearly scaled over this period range.

As recommended by FEMA (2012), this period range should be from T_{min} to T_{max} , where T_{max} is defined as twice the maximum of T_1^X and T_1^Y and T_{min} is the least of $0.2T_1^X$ and $0.2T_1^Y$. This gives a minimum period range (with the fundamental periods taken from table 3) of 0.17 s to 3.62 s. Also, to get a period range that includes at least 90% of the modal mass, the minimum period considered must be 0.14 s. The adopted period range is then 0.1 s to 4.0 s.

Tables 4 and 5 show the selected accelerograms and their scale factors. They also show the mean and standard deviation for the ratios of the scaled spectrum, S_{TH} , over the design spectrum, S_a . These ratios allows to quantify how well the selected accelerograms match the design spectrum. It shows that Atkinson’s accelerograms yield a more conservative spectrum with larger mean S_{TH}/S_a ratios while NGA West2’s match it more closely with smaller standard deviations of the S_{TH}/S_a ratios.

Table 3. Modal information for the core wall of the symmetrical building

Mode number	Period (s)	Cumul. Modal Mass Ratio (%)	
		Lat. EW	Lat. NS
1	1.81	0.00	65.84
2	1.70	71.07	65.84
3	0.44	88.30	65.84
4	0.34	88.30	87.88
5	0.20	93.59	87.88
6	0.14	93.59	94.72

5.6. Gravity loads and damping

In a technical report by the PEER’s Applied Technology Council Applied Technology Council (ATC) (2010), the recommended load case for gravity loads in a NLTHA is:

[8] $1.0D + 0.4 \times 0.5 \times L = 1.0D + 0.2L$

This load combination is the expected gravity load, which include the nominal dead load D and a fraction of the nominal live load L . The dead load includes the structure self-weight, the architectural finishes, and the permanent equipment. The live load is reduced to account for the low probability that the nominal live load occurs throughout the building ($\times 0.4$), and simultaneously with the earthquake load ($\times 0.5$). For linear and non linear dynamic analyses, gravity loads are applied to each column-slab node intersection according to tributary surfaces. The remaining load is applied at each core wall section’s centre. This load is used to determine dynamic lateral loads and to quantify the inelastic response of the wall.

For response spectrum analyses, damping is integrated using the NBCC’s 5% damped spectrum (National Research Council Canada 2010). Non linear models can capture a portion of the damping

Table 4. Selected artificial accelerograms from Atkinson's database

Accelerogram	Mag.	Dist.	SF §	Mean †	Std ‡
East-7d1-16-17	7.0	20.6 km	0.614	1.02	0.153
East-7d1-17-18	7.0	20.6 km	0.652	1.04	0.193
East-7d1-18-16	7.0	20.6 km	0.578	1.04	0.223
East-7d1-21-19	7.0	20.1 km	0.538	1.03	0.183
East-7d1-26-27	7.0	19.6 km	0.545	1.05	0.257
East-7d1-27-25	7.0	19.6 km	0.588	1.05	0.230
East-7d1-28-29	7.0	17.0 km	0.526	1.04	0.193
East-7d2-01-02	7.0	41.6 km	1.00	1.04	0.220
East-7d2-03-01	7.0	41.6 km	0.989	1.05	0.240

§ Scale Factor

† Mean of the S_{TH}/S_a ratio for the scaled accelerogram‡ Standard deviation of the S_{TH}/S_a ratio for the scaled accelerogram**Table 5.** Selected historical accelerograms from NGA-West2 database

Earthquake	RSN	Mag.	SF §	Mean †	Std ‡
Loma Prieta, 1989	767	6.93	0.56	0.979	0.137
Northridge, 1994	987	6.69	0.85	0.980	0.168
Northridge, 1994	1021	6.69	3.71	0.955	0.164
Chi-Chi, 1999	2954	6.20	2.82	0.974	0.163
Parkfield, 2004	4135	6.00	2.77	1.028	0.133
Parkfield, 2004	4137	6.00	2.33	0.965	0.153
Darfield, 2010	6949	7.00	2.39	1.01	0.145

§ Scale Factor

† Mean of the S_{TH}/S_a ratio for the scaled accelerogram‡ Standard deviation of the S_{TH}/S_a ratio for the scaled accelerogram

1 trough hysteresis and associated energy dissipation. A Rayleigh damping model proportional to the
 2 initial stiffness and mass matrices is used with a damping ratio $\xi = 2\%$. This value is representative
 3 of what is measured during dynamic tests of concrete structures. Two periods are used to calculate
 4 Rayleigh's coefficients, the fundamental period and the last period required to obtain 90% of the modal
 5 mass.

6. Analyses results

7 Figures 7, 8 and 9 present non linear transient and RSA results for the symmetric 12 storeys RC
 8 building when torsion is not considered. Figures 10, 11 and 12 present results for the same symmetric
 9 building when torsion is considered. Finally, figures 13, 14 and 15 present results for the offset core

Fig. 7. Lateral displacements in the symmetric building without torsion

Fig. 8. Lateral inter-story drifts in the symmetric building without torsion

1 building, taking into account torsion. Results are all presented for a corner column (column F6 in
2 figures 2 and 4) to capture the maximum torsional effect.

(a) Coupled walls direction

(b) Cantilever walls direction

Fig. 9. Absolute value of calculated forces in the symmetric building without torsion

6.1. Non linear time history analyses

Figures 7 to 15 present non linear time history analysis results labelled NLA. It refers to the mean results obtained with 7 accelerogram pairs from NGA West2 database (Pacific Earthquake Engineering Research Center 2014) and 9 pairs from Atkinson's ground motion database (Atkinson 2015). These figures also display a shaded area indicating the range ± 1 standard deviation of the series. This allows for a typical representation of the variation of results. Non linear analyses that account for the

Fig. 10. Lateral displacements in the symmetric building including accidental torsion

inelastic deformation profile of the SFRS is the default method recommended by Canadian standards (CSA 2014) to calculate seismic forces in the GLRS. It is the most rigorous analysis method used in this project and these results are used as a reference to which other methods are compared. Thus the shaded zone of the NLA curve defines the target values. For the studied buildings, a concentration of displacements and forces is visible near the base and reflect the formation of plastic hinges. This is particularly obvious in the drift and forces diagrams (Figs. 8, 9, 11, 12, 14 and 15).

6.2. Simplified analysis method from CSA A23.3-14

Curves corresponding to the simplified analysis method from CSA A23.3-14 are simply labelled CSA. They all have the same relative inter-story drift ratios (Figs. 8, 11 and 14) as it is the starting point for the analysis (Fig. 1). The drift profile shape is better represented in the cantilever wall direction than in the coupled wall direction as compared to the target values (shaded zone of NLA). This was to be expected because the A23.3-14 design drift profile has been derived from non linear analyses of cantilever walls. Indeed, coupled walls, under lateral seismic loads, show a drift profile with a distinctive bow near the base where deformations are larger as displayed in the NLTHA results. Furthermore, inverse leaning of the walls caused by moments in the coupling beams reduces the elastic portion of lateral displacement of the wall to being negligible. This cause the upper stories drift to be smaller than for a cantilever wall (White and Adebar 2004). Therefore, for the symmetric building, the drift is quite well estimated in the cantilever direction, while in the coupled direction, it is underestimated in the bow zone and overestimated in the upper stories. Both the displacements and base internal forces are conservatively assessed. In the upper storeys, figures 9, 12 and 15 show that shear forces are slightly underestimated at some points. However, the maximum bending moments for each story are in the lower bound of the target value range.

Fig. 11. Lateral inter-storey drifts in the symmetric building including accidental torsion

6.3. Response spectrum analysis with GLRS nearly null stiffness

The results, labelled "GNS" in the figures, are obtained by reducing the GLRS stiffness by a hundredth ($F_{sr} = 10^{-2}$) of its initial value as explained in section 4. This analysis method yields drift and displacement profiles which better represent the characteristic bow shape of the coupled walls drift than the CSA method. However, because the concentration of deformation towards the base due to inelastic deformation in the wall is not taken into account, it gives a lower bound estimate of internal forces and displacements. For the symmetric building, when torsion is not considered, this is the method that gives the smallest base forces and top displacements. Results including torsion show that GNS yields forces and displacements that are in the center of the reference range defined by NLTHA results. This is true for both the symmetric and offset core buildings.

6.4. Linear plastic hinge method

An appropriate reduction of the core wall elastic modulus in the plastic hinge region can provide results that better reflect the inelastic deformation profile. In figures 7 to 15, the results labelled LPH refers to analyses similar to GNS but including a core wall elastic modulus reduction of $E_{eff} = 0.35E_c$ for the two first stories while the other stories are reduced to a value of $E_{eff} = 0.7E_c$. The shear modulus is also reduced by the same factors because it is calculated by the software. This reduction is a simple way to account for the plastic hinge deformations effects on lateral displacements and member forces. These effects can be clearly seen in the displacement and drift profiles. A trial and error procedure was used in which the elastic modulus reduction factor was modified until the RSA base forces were within the NLTHA range when neglecting accidental torsion (Figs. 9). When torsion is included, the GNS with LPH gives a higher bound estimate of the seismic forces (Figs. 12 and 15). For the offset core building, seismic forces in the GLRS are far more important than those computed from the GNS without LPH. This high variability might explain in part why unsymmetrical buildings, like this offset core building, are not recommended in high seismic zones. Their behaviour is difficult to assess

Fig. 12. Absolute value of calculated forces in the symmetric building including accidental torsion

adequately with common analysis methods and unsymmetrical buildings often display unsatisfactory performance. This work demonstrates, once again, the importance of designing symmetrical buildings and to reduce torsional sensitivity to get the best seismic performance.

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Fig. 13. Lateral displacements in the offset core building including torsion

Fig. 14. Lateral inter-story drifts in the offset core building including torsion

7. Translational and rotational displacement demands on GLRS

One of the main advantage of the proposed method is its ability to capture all translational and rotational displacement patterns of the different elements constituting the GLRS, not only the columns

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Fig. 15. Absolute value of calculated forces in the offset core building including torsion

1 shear displacements. Figure 16 shows the deformed shape of the Digicel building located in Port-au-
2 Prince, Haiti computed from a finite element simulation of the M7 2010 Haiti earthquake. Figure 16
3 is obtained from a particular time step during a non linear time history analysis performed with the
4 Seismostruc program (Seismosoft 2013b). The 12 storey frame-wall structure was one of the few to
5 survive the moment magnitude 7 January 12, 2010 Haiti earthquake. The earthquake epicenter was just
6 15 km away from the building. The columns in the top six storeys suffered mostly concrete spalling

Fig. 16. Predicted translational and rotational displacements from non linear time history analysis of the Digicel Buildings damaged during the 2010 Haiti earthquake reproduced from Boulanger, Paultre and Lamarche (2013).

at their ends. However, girders suffered significant damage and yielding at their connections to the structural walls in the top 6 storeys. These damage can be explained by the rotations imposed by the structural walls at their connections to the girders. Additional information can be found in Boulanger, Paultre and Lamarche (2013). The ability of the proposed method to capture demand from translational and rotational displacement compatibility at the connections between the structural wall and the girders is obvious in Fig. 16.

Our study concentrated on fixed based structures. Obviously, a complete model should include the structure below grade. Foundation movements and structures below grade will lengthen the period of the structure and usually would reduce curvature demands at the base of the walls. However, it is important to mention that the CSA A23.3 impose that the minimum curvature demand on all columns or walls over the plastic hinge length region of the SFRS shall not be taken less than the curvature demand associated with the inelastic rotational demands on the SFRS.

8. Conclusions

This paper presents a new simple and reliable analysis method to calculate seismic forces in elements not part of the SFRS of RC buildings. It also assess the new method's validity by comparing its results to NLTHA results. Additionally, it uses the NLTHA results to assess the A23.3-14 simplified method. From the analysis results, it is concluded that :

1. The proposed method (GNS), is simple and convenient to implement in a conventional design procedure.
2. The GNS method gives a better representation than that of A23.3-14 for the drift profile of coupled walls because it models them explicitly. Moreover, it provides the ability to maintain

- translational and rotational displacement compatibility between the shear walls and the gravity load resisting system.
3. The GNS results represents an accurate estimate of what would be obtained from a non linear transient analysis carried out in accordance with NBCC 2010 and A23.3-14 when torsion is included.
 4. If the proposed method is combined with a reduction of the walls elastic modulus in the plastic hinge zone to represent the inelastic deformation profile of the SFRS (LPH), higher bound displacements and forces are produced at the base of the structure as compared to non linear transient analyses.
 5. As for the CSA method (CSA A23.3-14, art. 21.11.2.2), it conservatively estimate seismic forces in columns in the plastic hinge zone, and gives a lower bound estimate in the upper stories.

This new method is one of many that can be used to calculate the demand placed on GLRS. Its advantage is that it uses only one finite element model of buildings that are designed according the NBCC and thus does not increase the cost of analysis and design but indeed reduces it. In addition, the method can predict the demand on columns and beams, accounting for all interaction with the SFRS. Effects of underground storeys and foundation displacements have not been addressed in this paper but it is known that they have significant effects on the seismic demand. Inertial effects, frequency dependent soil properties, stiffness of the underground structures, intensity of excitations are all important parameters that need to be accounted for. Modelling of these effects is not simple and some guidelines can be found in (PEER 2010). This paper presents a framework that could be expanded to account for these effects for determining displacement and rotational demands on GLRS that can be effectively used in design offices.

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References

- ACI committee 318 (2014), *318-14: Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, MI.
- Adebar, P. and Dezhdar, E. (2015), Effective stiffness for linear dynamic analysis of concrete shear wall buildings : CSA A23.3 - 2014, in 'The 11th Canadian Conference on Earthquake Engineering', Canadian Association for Earthquake Engineering, Victoria, BC, Canada.
- American Society of Civil Engineers (2013), *Minimum Design Loads for Buildings and Other Structures*, asce/sei 7-10 edn, Reston, VA.
URL: <http://ascelibrary.org/doi/abs/10.1061/9780784412916>
- Anagnostopoulos, S., Kyrkos, M. and Stathopoulos, K. (2015), 'Earthquake induced torsion in buildings: critical review and state of the art', *Earthquakes and Structures* **8**(2), 305–377.
- Applied Technology Council (ATC) (2010), Modeling and acceptance criteria for seismic design and analysis of tall buildings, Report No. ATC-72-1, Technical report, Pacific Earthquake Engineering Research Center (PEERC), Redwood City, CA.

- 1 Atkinson, G. M. (2015), 'Engineering Seismology Toolbox', www.seismotoolbox.ca. Accessed:
2 2015-02-02.
- 3 Bazargani, P. (2014), Seismic demands on gravity-load columns of reinforced concrete shear wall
4 buildings, PhD thesis, University of British Columbia, Vancouver.
- 5 Beyer, K., Dazio, A. and Priestley, M. J. N. (2008), 'Inelastic wide-column models for U-shaped rein-
6 forced concrete walls', *Journal of Earthquake Engineering* **12**(sup1), 1–33.
7 **URL:** <http://dx.doi.org/10.1080/13632460801922571>
- 8 Boulanger, B., Lamarche, C.-P., Proulx, J. and Paultre, P. (2013), 'Analysis of a damaged 12-storey
9 frame-wall concrete building during the 2010 Haiti earthquake Part I: Dynamic behaviour assess-
10 ment', *Canadian Journal of Civil Engineering* **40**(8), 791–802.
11 **URL:** <http://www.nrcresearchpress.com/doi/abs/10.1139/cjce-2012-0098>
- 12 Boulanger, B., Paultre, P. and Lamarche, C.-P. (2013), 'Analysis of a damaged 12-storey frame-wall
13 concrete building during the 2010 Haiti earthquake Part II: Nonlinear numerical simulation', *Cana-
14 dian Journal of Civil Engineering* **40**(8), 803–814.
15 **URL:** <http://dx.doi.org/10.1139/cjce-2012-0099>
- 16 Computer and Structures, Inc. (2010), 'ETABS nonlinear version 9.7.1'.
- 17 CSA (2014), *Design of Concrete Structures : CAN/CSA-A23.3-14*, Canadian Standards Association.
- 18 Dezhdar, E. (2012), Seismic response of cantilever shear wall buildings, PhD thesis, University of
19 British Columbia.
- 20 Dezhdar, E. and Adebar, P. (2015), Estimating seismic demand on concrete shear wall buildings, in
21 'The 11th Canadian Conference on Earthquake Engineering', Canadian Association for Earthquake
22 Engineering, Victoria, BC, Canada.
- 23 Elwood, K. J. (2013), 'Performance of concrete buildings in the 22 february 2011 christchurch earth-
24 quake and implications for canadian codes', *Canadian Journal of Civil Engineering* **40**(1), 1–18.
- 25 FEMA (2012), Seismic performance assessment of buildings report no. p58-1, volume 1 - methodol-
26 ogy, Technical report, Applied Technology Council (ATC), Redwood City, CA, USA.
- 27 Filippou, F. C., Popov, E. P. and Bertero, V. V. (1983), Effects of bond deterioration on hysteretic
28 behavior of reinforced concrete joints, Technical report, Earthquake Engineering Research Center.
- 29 Huang, X. and Kwon, O.-S. (2015), 'Numerical models of RC elements and their impacts on seismic
30 performance assessment', *Earthquake Engineering & Structural Dynamics* **44**(2), 283–298.
31 **URL:** <http://dx.doi.org/10.1002/eqe.2471>
- 32 Kara, I. F. and Dundar, C. (2009), 'Prediction of deflection of reinforced concrete shear walls', *Ad-
33 vances in Engineering Software* **40**(9), 777–785.
34 **URL:** <http://www.sciencedirect.com/science/article/pii/S0965997809000179>
- 35 Mander, J., Priestley, M. and Park, R. (1988), 'Theoretical stress-strain model for confined concrete',
36 *Journal of Structural Engineering* **114**(8), 1804–1826.
37 **URL:** [http://dx.doi.org/10.1061/\(ASCE\)0733-9445\(1988\)114:8\(1804\)](http://dx.doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804))
- 38 Menegotto, M. and Pinto, P. (1973), Method of analysis for cyclically loaded rc plane frames including
39 changes in geometry and non-elastic behavior of elements under combined normal force and bend-
40 ing, in 'IABSE symposium on resistance and ultimate deformability of structures acted on by well
41 defined repeated loads', pp. 15–22.

- 1 Mitchell, D., DeVall, R. H., Saatcioglu, M., Simpson, R., Tinawi, R. and Tremblay, R. (1995), 'Damage
2 to concrete structures due to the 1994 Northridge earthquake', *Canadian Journal of Civil Engineer-*
3 *ing* **22**(2), 361–377.
4 **URL:** <http://www.nrcresearchpress.com/doi/abs/10.1139/l95-047>
- 5 Mitchell, D. and Paultre, P. (2006), *Concrete design handbook*, Cement Association of Canada, Ottawa,
6 ON, chapter 11 Seismic design.
- 7 Monti, G., Nuti, C. and Santini, S. (1996), 'Cyrus-cyclic response of upgraded sections', *University of*
8 *Chieti, Italy*.
- 9 National Research Council Canada (2010), *National Building Code of Canada 2010*, Canadian commis-
10 sion on building and fire codes, Ottawa, ON.
- 11 Pacific Earthquake Engineering Research Center (2014), 'PEER ground motion database', [http://](http://ngawest2.berkeley.edu/)
12 ngawest2.berkeley.edu/. Accessed: 2015-09-01.
- 13 Paulay, T. (1986), 'The design of ductile reinforced concrete structural walls for earthquake resistance',
14 *Earthquake Spectra* **2**(4), 783–823.
- 15 PEER (2010), Guidelines for performance-based seismic design of tall buildings, Technical Report
16 PEER 2010/05, Pacific Earthquake Engineering Research Center University of California, Berkeley,
17 CA.
- 18 Pekau, O. and Guimond, R. (1990), 'Accidental torsion in yielding symmetric structures', *Engineering*
19 *Structures* **12**(2), 98 – 105.
20 **URL:** <http://www.sciencedirect.com/science/article/pii/014102969090014J>
- 21 Ramadan, O., Mehanny, S. and Mostafa, A. (2008), Revisiting the 5% accidental eccentricity provi-
22 sion in seismic design codes for multi-story buildings, in '14th World Conference on Earthquake
23 Engineering', Beijing, China.
- 24 Seismosoft (2013a), *SeismoStruct User Manual*.
- 25 Seismosoft (2013b), 'SeismoStruct v6.5 - A computer program for static and dynamic nonlinear anal-
26 ysis of framed structures'. available from <http://www.seismosoft.com>.
- 27 White, T. W. and Adebar, P. (2004), Estimating rotational demands in high-rise concrete wall buildings,
28 in '13th World Conference on Earthquake Engineering'.