The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

ESTIMATING SEISMIC DEMANDS ON CONCRETE SHEAR WALL BUILDINGS

Ehsan DEZHDAR
Structural Engineer, Glotman Simpson Consulting Engineers
Email: edezhdar@glotmansimpson.com

Perry ADEBAR
Professor of Structural Engineering, The University of British Columbia
Email: adebar@civil.ubc.ca

ABSTRACT: The results from numerous nonlinear response history analysis (NLRHA) was used to develop simplified empirical methods to estimate maximum top-wall displacement, maximum base and mid-height curvatures, and maximum inter-story drift using only (linear) response history analysis (RSA). The effective flexural rigidity (stiffness) ratio (EI_e/EI_g) used to estimate top-wall displacement was found to reduce from 1.0 to 0.5 as the ratio of elastic demand to bending strength of the wall increases from 1.0 to 3.5. The level of axial compression force applied to the wall is less important. When the increased flexibility due to flexural cracking is accounted for, the maximum curvature demands near mid-height of the wall were found to be less than a typical value of yield curvature and thus a limited amount of ductility is needed to tolerate the bending demands from higher modes. The current Canadian Concrete Code CSA A23.3 method for estimating inelastic rotation at the base of concrete walls was found to be conservative. Approaches for refining these calculations are proposed. Envelopes of maximum inter-story drifts are proposed for evaluating the seismic demands on gravity-load frames.

1. Introduction

Nonlinear response history analysis (NLRHA) is the most accurate way to estimate the seismic demands on a shear wall building. While this approach is commonly used in the so-called performance-based seismic design of high-rise shear wall buildings in the United States, it is not commonly used for design in Canada. One disadvantage of NLRHA is that the results are dependent on what ground motions are selected, how the ground motions are scaled, and what non-linear hysteretic models are used to represent the response of the structure.

An alternate approach to using NLRHA for design is to use linear dynamic – response spectrum – analysis (RSA) combined with simple empirical methods to estimate the nonlinear demands. For example, the maximum displacement at the top of a cantilever shear wall can be accurately estimated from RSA if an appropriate effective flexural rigidity EI_e is used to account for the nonlinear effects due to cracking. Once the maximum top wall displacement is known, it can be used as the input to simple empirical methods that provide estimates of the nonlinear demands on the shear walls, as well as the nonlinear demands on the gravity-load frame.

The expected level of damage in a shear wall is directly related to the maximum strain demands. Simple empirical methods can be used to convert top wall displacement into maximum inelastic rotational demand at the base of the wall, as well as maximum curvature demand and maximum strain demand. The seismic demands on the gravity-load frame in a shear wall building, e.g., curvature demands on columns or rotational demands on slab-column connections, can be determined from the envelope of maximum inter-story drifts demands over the height of the building. While RSA can be used to estimate the maximum top wall displacement, it cannot be used to estimate the envelope of maximum interstory drifts as it does not account for the nonlinear distribution of deformations. A large portion of the nonlinear deformation of a cantilever shear wall are concentrated in the plastic hinge region at the base of the wall.

In this paper, simplified empirical methods are presented for estimating the seismic deformation demands in shear wall buildings. A brief summary is first presented about how the NLRHA was done in order to generate the data needed to develop the empirical methods. This includes the example shear wall buildings that were used, the ground motions that were selected and how the concrete shear walls were modeled.

2. NLRHA of Example Buildings

2.1. Shear Wall Buildings

Thirteen different cantilever shear wall buildings were studied. The main parameters were the height of the building and the strength of the shear walls, which may be defined by the parameter R_g = the ratio of elastic bending moment demand at the base of the walls calculated using the uncracked flexural stiffness EI_g to the nominal flexural strength M_n. The study included three 10-story buildings with R_g = 1.7, 2.6, and 4.2; one 20-story building with R_g = 4.0; four 30-story buildings with R_g = 1.4, 2.4, 3.1, and 4.3; one 40-story wall with R_g = 4.4; and four 50-story walls with R_g = 1.4, 2.1, 2.4, and 4.1. The thirteen buildings were designed according to the requirements of CSA A23.3-04. The amount of vertical reinforcement at the zones was determined considering the level of axial compression force due to gravity loads and the target R_g. The full description of the shear walls can be found in Dezhdar (2012).

2.2. Ground Motions

The input ground motions in this study were scaled to the uniform hazard spectrum (UHS) for Vancouver BC for site class C. Eighty ground motions with moment magnitude between 6.5 and 8.0 and closet source to site distance between 0.5 and 50 km were included. A comprehensive study was carried out to investigate the influence of ground motion selection and scaling on various demand parameters (Dezhdar and Adebar, 2015). Time history results presented in this study correspond to the ground motions that were matched to the target UHS over a range of periods wider than $0.2T_1$ to $1.5T_1$, where T_1 is equal to 1.0, 2.0, 3.0, 4.0, and 5.0 seconds for 10, 20, 30, 40, and 50 story shear walls, respectively.

2.3. Nonlinear Modeling of Shear Walls

The nonlinear response history analysis of the 13 shear walls was done using OpenSees (2008) with the trilinear hysteretic bending moment - curvature relationship presented in Fig. 1. The hysteretic model was verified by making comparisons with experimental results from two large-scale tests of slender shear walls with flanged and rectangular cross sections (Adebar and Ibrahim, 2002). Details of the hysteretic model can be found in Dezhdar (2012). The parameters that define the trilinear model were calculated at each floor considering the variation of axial compression. A force element was defined at each floor level to model the vertical spread of plasticity in the walls. The base was assumed to be fixed, and shear deformations were not considered in the analytical model. Rayleigh damping was assumed with mass proportional and initial stiffness matrixes. A damping ratio of 3% was assigned for the first and third modes.

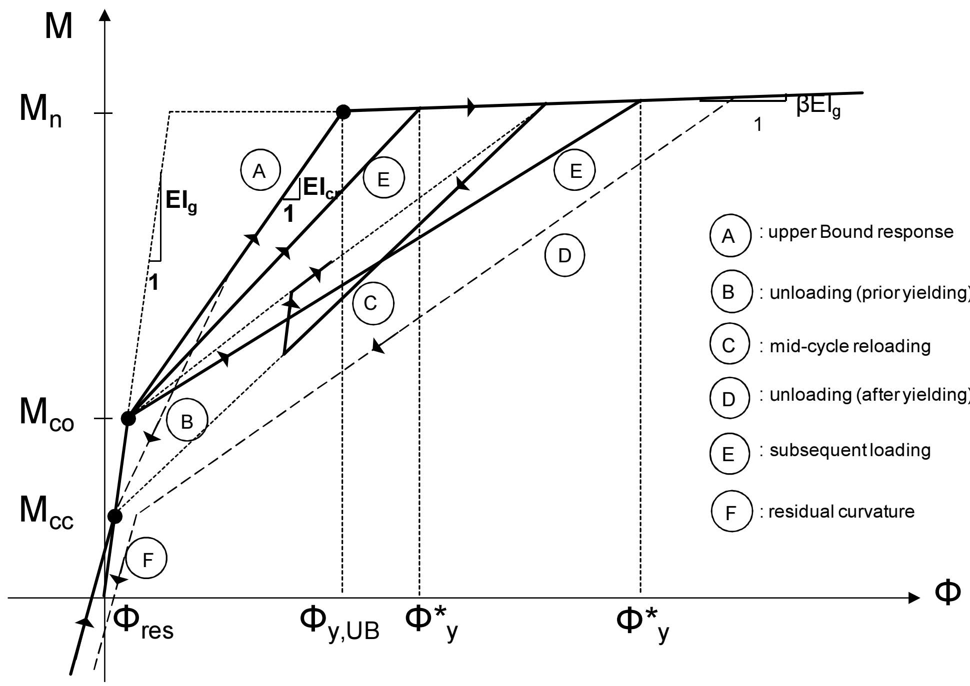


Fig 1 – Trilinear Hysteretic Bending Moment - Curvature Relationship Used In the Nonlinear Response History Analysis (NLRHA).

3. Top-Wall Displacement

If the correct effective stiffness (effective flexural rigidity EI_e) is used, linear response spectrum analysis (RSA) will give exactly the correct top wall displacement. The appropriate reduction in effective stiffness can be expressed in term of the parameter α = the ratio of effective flexural rigidity EI_e (used in RSA) to the initial uncracked (gross section) flexural rigidity EI_g.

The current Canadian concrete code CSA A23.3-04 contains the follow expression for the ratio:

$$\alpha = \frac{EI_e}{EI_g} = 0.6 + \frac{P}{f'_c A_g} \tag{1}$$

where P is the axial force at the base of the wall due to gravity loads. This expression reflects the fact that larger axial compression results in a more linear load – deformation response.

For each building, of given height and wall strength, α was determined so that the results from the RSA would give exactly the mean top-wall displacement determined from NLRHA. A stiffness reduction factor of 1.0 was assumed as the first guess and then was reduced iteratively until the best match for top-wall displacement was achieved. Lower reduction factors are needed for walls with higher nonlinear action. Fig.2 plots the reduction factor α as a function of the strength ratio R for the 13 example shear wall buildings. Also shown in the plot is the proposed equation for the ratio α as a function of the force ratio R_g.

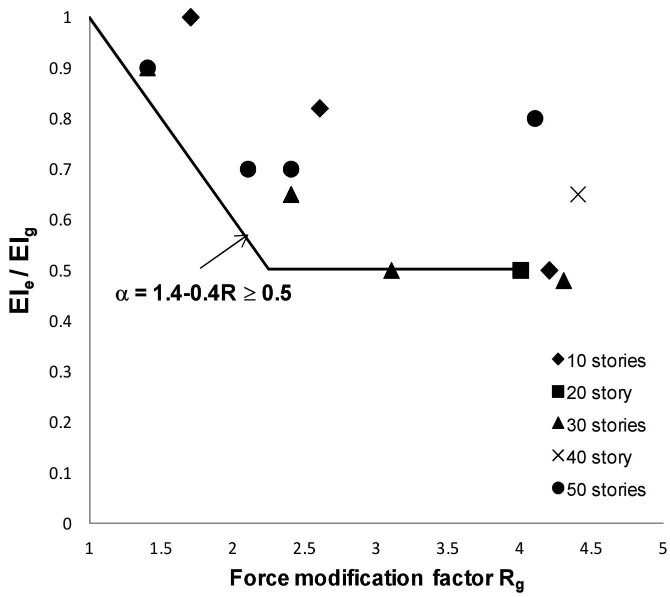


Fig. 2 – Stiffness Reduction Factor as a Function of R_g for the Thirteen Example Buildings.

4. Maximum Wall Curvature at Base

According to CSA A23.3-04, the curvature demand ϕ_d at the base of a shear wall can be determined from the following expression:

$$\phi_d = \phi_y + \frac{2}{l_w}\frac{\Delta_t(1-\frac{1}{R})}{h_w - 0.25 l_w} \tag{2}$$

Where the assumptions used to develop Eq. (2) include: (1) the elastic portion of the top-wall displacement demand is estimated as being equal to the total displacement demand Δ_t divided by the flexural strength ratio *R,* where *R* = elastic bending moment demand to the actual bending strength of the wall. (2) An upper-bound estimate of the yield (elastic) curvature is taken equal to 0.004 divided by the wall length l_w. (3) A lower-bound estimate of the plastic hinge length of the wall is taken as $0.5l_w$. Additional information about the derivation of Eq. (2) is given in Adebar et al., 2005. h_w in Eq. (2) is the overall wall height.

Fig. 3 compares the predicted base curvature demand from Eq. (2), i.e., the CSA A23.3-04 approach, with the NLRHA results from the current study. Also shown in Fig. 3 is the prediction from two other models: the M1 model developed for predicting the mean curvature demand, and the M2 model for predicting the mean plus one standard deviation curvature demand. Model 1 is given by Eq. (3) combined with Eq. (4) for C, while Model 2 is given by Eq. (3) combined with Eq. (5).

$$\phi_d = C\frac{\Delta_t}{h_w l_w} \tag{3}$$

$$C_{mean} = 1.8 - 0.017\frac{h_w}{R} \tag{4}$$

$$C_{mean+SD} = 2.8 - 0.022\frac{h_w}{R} \tag{5}$$

Note that the force ratio R in Eqs. (4) and (5) is based on the effective stiffness of the wall EI_e, rather than the uncracked stiffness EI_g. As a result, the values of R are smaller than the values of R_g presented above. Additional information about this model is given in Dezhdar (2012).

Fig.3 indicates that the CSA A23.3-04 prediction of the base curvature is between the mean and mean plus one standard deviation NLRHA results for the 10-story buildings, but it is consistently higher than the mean plus one standard deviation NLRHA results for the 30 and 50-story buildings. The CSA A23.3-04 approach can be refined by revising the assumption for the elastic (yield) curvature and/or the elastic portion of the top-wall displacement demand. Further information is given in Dezhdar (2012).

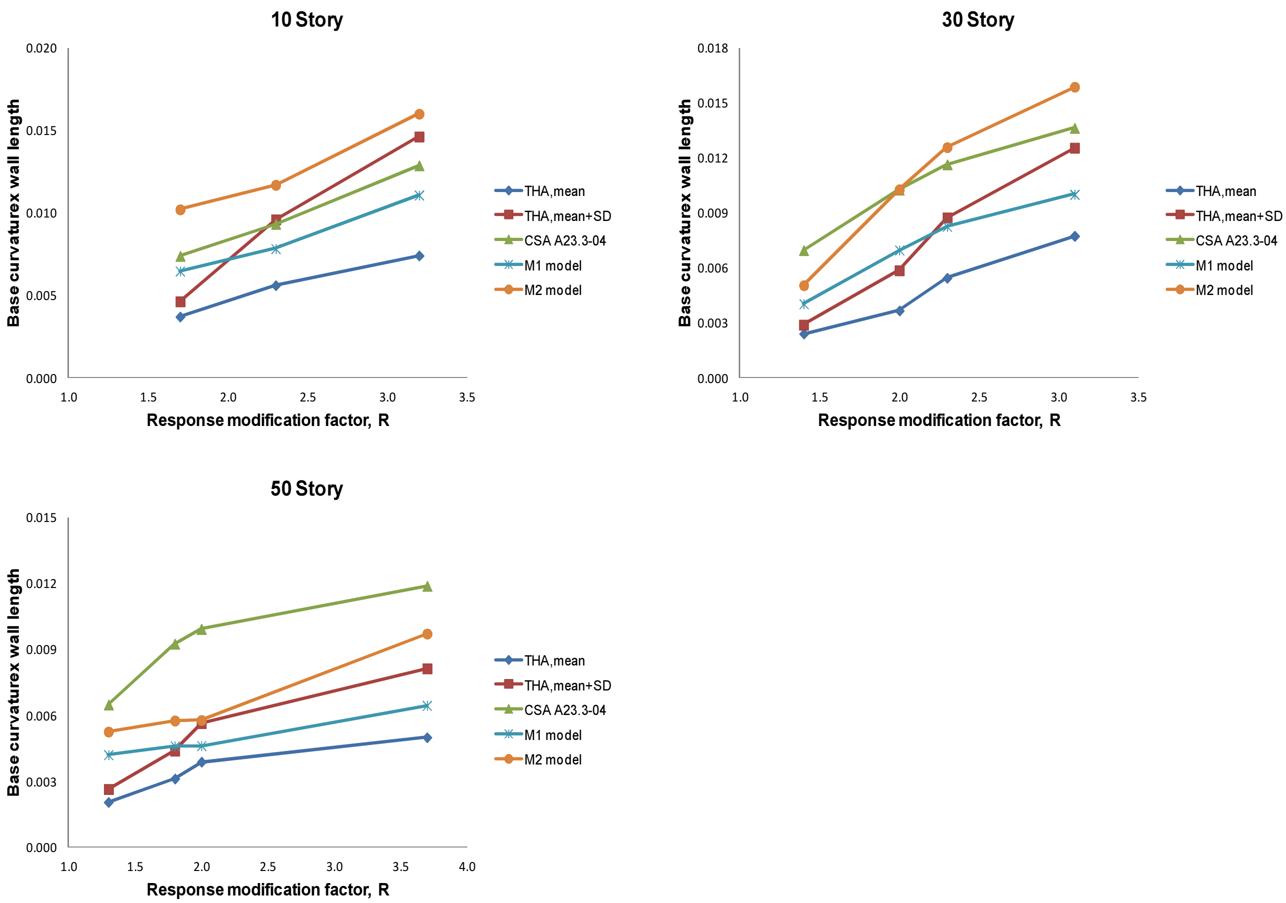


Fig. 3 – Comparison of Base Curvature Demands from Different Models with the Mean and Mean Plus One Standard Deviation Results from Time History Analysis (Denoted as THA) for 10 to 50 Story Buildings; Note THA = NLRHA.

5. Curvature Demand at Mid-height of Building

The current procedure for designing concrete shear wall buildings in Canada assumes there will be a plastic hinge zone at the base of the wall and the rest of the wall will remain essentially elastic. A number of previous studies using NLRHA (Panneton et al, 2006; Moehle et al, 2007; Panagiotou and Restrepo, 2009) have shown that large bending moments may develop near the mid-height of cantilever shear walls due to higher mode bending moments, and thus concrete shear walls may experience significant flexural yielding near mid-height. As ductile detailing is typically not provided near mid-height, significant curvature demands at mid-height is of considerable concern.

One question that arises is to what extent does the variation of flexural strength of the shear walls near the mid-height of the building change the curvature demands near mid-height. Fig. 4 presents the mean curvature demand over the height of four different 50-story buildings together with the envelope of flexural capacity of the walls. Fig. 4 indicates that the base curvature demand increases significantly as the flexural strength of the wall is reduced near the base; however the mean mid-height curvature is much less sensitive to the flexural strength of the walls. For example, maximum mid-height curvatures for 50-story buildings with R = 1.8 and R = 3.7 are identical (0.133 rad/km for R = 1.8 versus 0.132 rad/km for R = 3.7). At the location that the maximum curvatures occur for the R = 1.8 building, the flexural strength of the walls with R = 1.8 is 1.5 times the flexural strength of the walls with R = 3.7. Therefore, it seems that increasing the flexural strength only slightly reduces the mid-height curvature demands, while it significantly reduces the maximum curvature demands at the base.

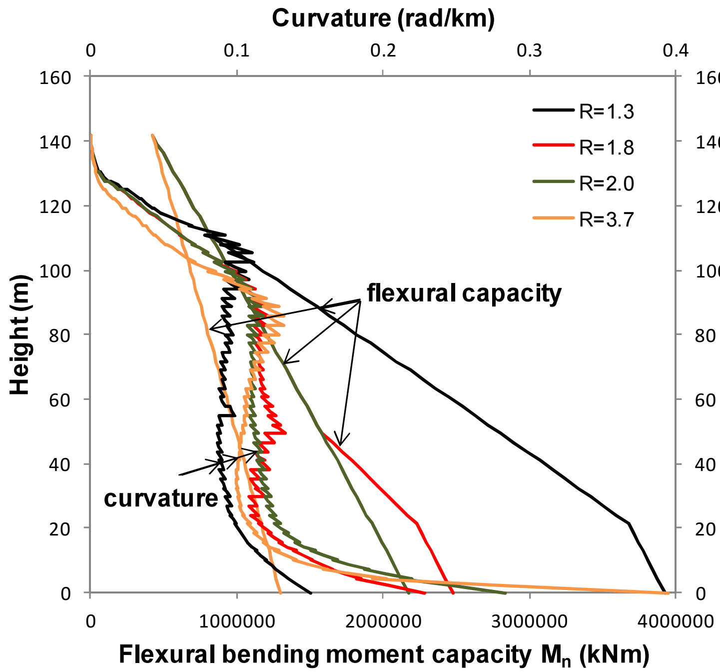


Fig. 4 – Relationship between Mid-Height Curvature Demand Envelopes and Flexural Capacity Envelopes for 50-Story Buildings with Different Flexural Capacities.

Table 1 shows the mean and the mean plus one standard deviation of mid-height curvature demands determined from NLRHA. The product of curvature and wall length gives the total strain variation across the wall, i.e., the sum of the maximum compression strain and the maximum tension strain with both taken as positive values. Table 1 indicates that the product of maximum mid-height curvature times the wall length varies from 0.0014 to 0.0023 for the mean results and varies from 0.0025 to 0.0042 for the mean plus one standard deviation results. This indicates that the strains are not very large at mid-height. The mean value of the mid-height curvature demand times the wall length for the thirteen buildings for the mean and mean plus one standard deviation NLRHA results is 0.0019 and 0.0034, respectively. Therefore if may be summarized that $0.002/l_w$ and $0.0035/l_w$ are appropriate values for mid-height curvature demands depending whether mean or mean plus one standard deviation of NLRHA results are used. Comparing these two values with the typical assumed range for the yield curvature of a wall ($0.0025/l_w$ to $0.004/l_w$) reveals that mid-height curvature demands in concrete shear walls are generally less than the assumed value for yield curvature. Because the strain demands are very low at mid-height, the only design requirement is to ensure the vertical tension reinforcement will yield prior to the concrete reaching a compression strain that may do damage.

Table 1 – Mean (μ) and mean plus one standard deviation ($\mu+\sigma$) results for mid-height curvature times wall length.

	No. of Stories	10			20	30				40	50				
$\Phi_{mid}.l_w$	R	1.7	2.3	3.3	2.7	1.3	2.0	2.3	3.1	3.6	1.3	1.8	2.0	3.7	Average
	μ (x 10^3)	1.8	2.0	2.1	2.3	1.4	2.0	1.9	2.1	1.8	1.5	1.8	1.9	2.3	**1.9**
	$\mu+\sigma$ (x10^3)	3.5	3.7	3.7	4.2	2.7	3.9	3.2	3.7	3.3	2.7	2.9	2.5	4.0	**3.4**

6. Inter-story Drift Envelopes

Establishing a simple model for estimating inter-story drift demand envelopes requires relating inter-story drifts at a number of points over the height to an appropriate demand parameter that can be easily estimated. The maximum inter-story drift at the top of the wall and at the mid-height of the wall are expressed as a function of top-wall displacement demand since, as it was shown in Section 3, the top-wall displacement can be accurately predicted using RSA. The relationship between maximum top-wall and mid-height inter-story drifts with the top-wall displacement demand was investigated by examining the time-history plots of the three demand parameters for individual ground motions. Fig. 5 summarizes the results for three 10, 30, and 50-story walls with R values of 3.2, 3.1, and 3.7, respectively.

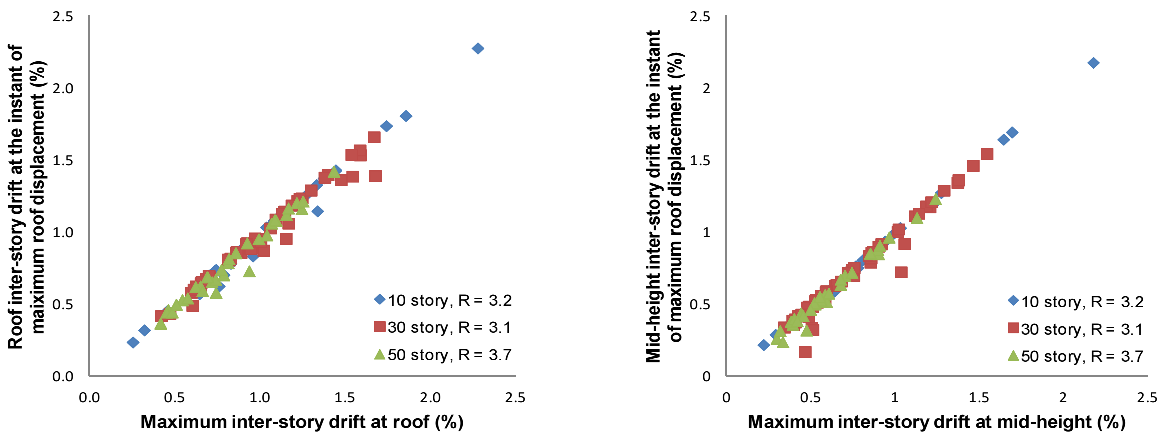


Fig. 5 – Top-wall (Roof) and Mid-height Inter-story Drifts at the Instant of Maximum Top-wall Displacement for the 10, 30, and 50-Story Buildings.

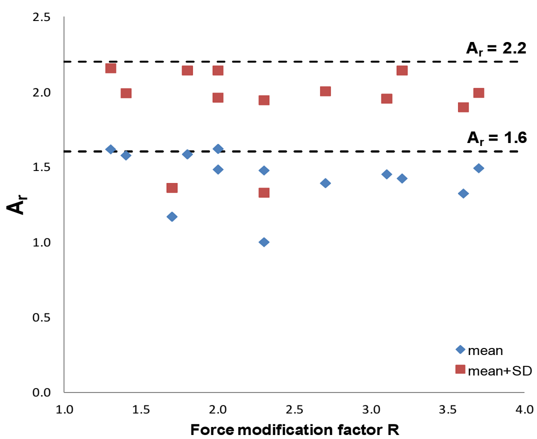
Fig. 5 indicates that the top-wall and mid-height inter-story drift demands at the instant of maximum top-wall displacement are very close to the maximum values during the ground motion record. The scatter is larger for the 30 and 50-story buildings than for the 10-story buildings. Fig. 5 also shows that both the top-wall and mid-height inter-story drifts are strongly correlated to the top-wall displacement demand. As a result, top-wall and mid-height inter-story drifts were expresses in terms of the top-wall displacement demand as follows:

$$(ID)_r = A_r \frac{\Delta_t}{h_w} \quad (6)$$

$$(ID)_m = A_m \frac{\Delta_t}{h_w} \quad (7)$$

where, $(ID)_r$ and $(ID)_m$ are the inter-story drift at the top of the wall and at the mid-height, Δ_t is the top-wall displacement demand, and h_w is the wall height. Parameters A_r and A_m were calculated using $(ID)_r$ and $(ID)_m$ values obtained from NLRHA and the top-wall displacement demand Δ_t from RSA using the stiffness reduction factors presented in Fig. 2. The computed mean and mean plus one standard deviation values of A_r and A_m as a function of the force ratio R for the thirteen walls are shown in Fig. 6.

Fig. 6 indicates that both A_r and A_m are relatively independent of R regardless of whether the mean or mean plus one standard deviation results are used. The dashed lines in these figures represent upper bound estimates of the two parameters, i.e., 1.6 and 2.2 for A_r and 1.3 and 1.8 for A_m corresponding to the mean and mean plus one standard deviation results, respectively. With the values of A_r and A_m known, the top-wall and mid-height inter-story drift demands can be determined from Eq. (6) and Eq. (7), respectively.



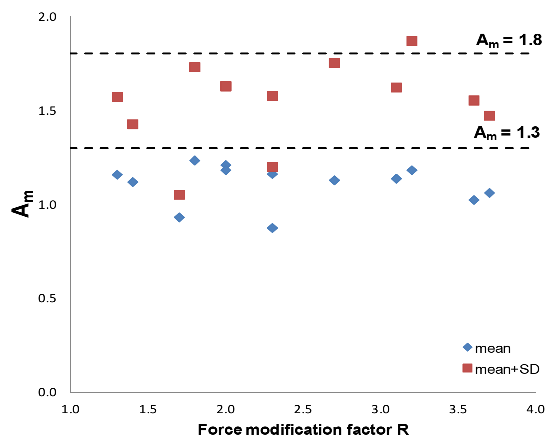


Fig. 6 – Variation of A_r and A_m Corresponding to the Mean and the Mean Plus One Standard Deviation Results from NLRHA as a Function of Wall Strength Ratio R for the Thirteen Buildings.

A proposed envelope for the inter-story drift demands over the height is shown in Fig. 7. This envelope is from the results of NLRHA where the shear walls are assumed to be supported on idealized infinitely rigid foundations at the base. In real buildings, there will be some inter-story drift at the base of the wall due to deformation of the foundation or due to curvatures in shear walls that extend several floors below the base. Fig.8 presents a further simplified inter-story drift envelope that includes significant inter-story drift at the base to account for the issues mentioned above as well as the inter-story drift due to shear strain in the wall.

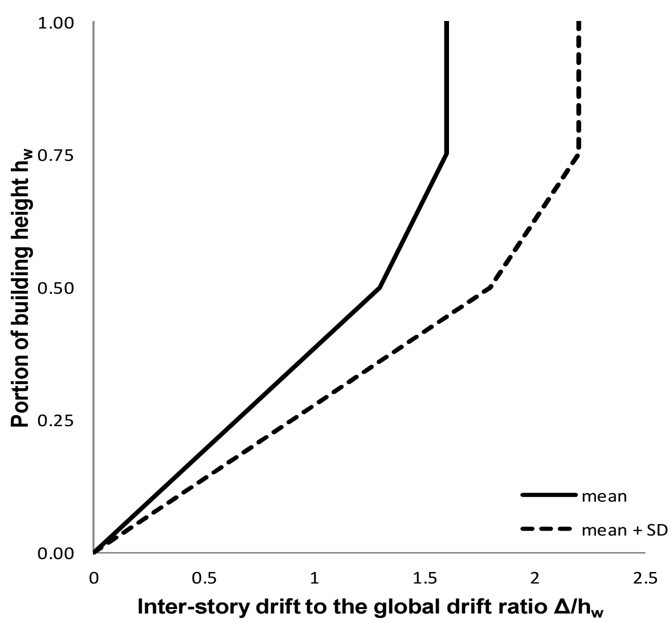


Fig. 7 – Proposed Inter-story Drift Envelopes for Cantilever Walls Supported by Perfectly Rigid Foundations at the Base.

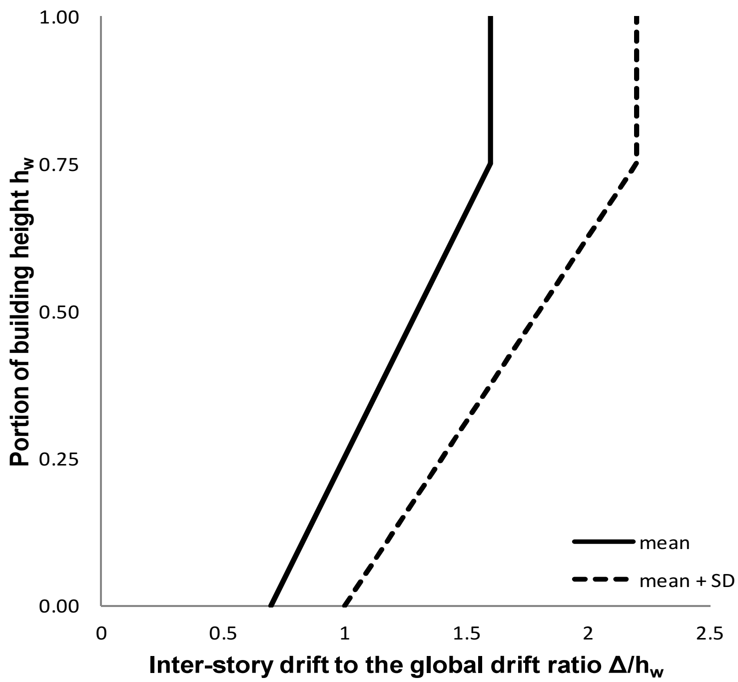


Fig. 8 – Simplified Inter-story Drift Envelopes Accounting for Inter-story Drifts at the Base.

7. Summary and Conclusions

Nonlinear response history analysis (NLRHA) was used to develop simplified empirical models to estimate the seismic demands in cantilever shear wall buildings. The study included 13 different buildings with different cantilever shear walls that were 10 to 50 stories high, and had a wide range of longitudinal reinforcement percentages and axial compression force levels. Mean top-wall displacement demands from NLRHA were used to determine the effective stiffness of cantilever shear walls. It was observed that stiffness reduction factor reduces from 1.0 to 0.5 as the elastic bending moment demand to wall strength ratio increases. Using the recommended reduction factors in a linear analysis such as response spectrum analysis (RSA) results in top-wall displacement demands that are similar to the mean displacement demands from time history analysis.

It was observed that the base curvature demand predicted by CSA A23.3-04 are generally higher than the mean plus one standard deviation results from NLRHA. A model was proposed for the base curvature demands corresponding to mean and mean plus one standard deviation NLRHA results. The proposed model relates base curvature demands to the global drift demand through a term that is a function of the wall height and force reduction factor. It was also observed that the intensity of mid-height curvature demand is relatively independent of the wall flexural strength, and mid-height curvature demand is generally $0.002/l_w$ (for mean NLRHA results) and $0.0035/l_w$ (for mean plus one standard deviation NLRHA results), where l_w is the wall length. This level of strain in the shear walls can easily be tolerated by simply ensuring that the vertical steel in the wall will yield before any crushing of the concrete in compression between the base of the wall and 75% of the wall height.

An important outcome of this study is the proposed inter-story drift demand envelopes for cantilever shear walls. Accurate estimates of the inter-story drift are important for assessing the seismic demands on the gravity-load frame, e.g., determining the rotational demands on slab-column connections, which directly influences the shear strength of the slab. Top-wall and mid-height inter-story drifts were expressed in terms of the global drift ratio (top-wall displacement divided by height of wall). It was observed that both the top-wall and mid-height inter-story drift demands are relatively independent of the flexural strength of the wall. Two different complete inter-story drift envelopes are proposed. The more complex one, which gives zero interstory drift at the base is only appropriate if the base of the wall has a very rigid support. The simpler envelope accounts for additional interstory drifts due to a non-rigid support and/or shear strains in the wall, neither of which is typically modelled in NLRHA.

8. References

ADEBAR, Perry, IBRAHIM, Ahmed, "Simple nonlinear flexural model for concrete shear walls", *Earthquake Spectra*, Vol. 18, No. 3, Aug. 2002, pp. 407-426.

ADEBAR, Perry, MUTRIE, James, and DEVALL, Ronald, "Ductility of Concrete Walls: the Canadian Seismic Design Provisions 1984 to 2004", *Can. J. of Civ. Eng.*, Vol. 32, No. 6, December 2005, pp. 1124-1137.

ATC 72, "Modeling and acceptance criteria for seismic design and analysis of tall buildings", Applied Technology Council, Redwood City, California, 2010.

CSA Committee A23.3 "Design of Concrete Structures: Structures (Design) – A National Standard of Canada" Canadian Standards Association 2004, Rexdale, Canada, 214 pp.

DEZHDAR, Ehsan, "Seismic Response of Cantilever Shear Wall Buildings", *Ph.D. Thesis*, The Univ. of British Columbia, December 2012.

DEZHDAR, Ehsan, and ADEBAR, Perry, "Influence of ground motion scaling on seismic response of concrete shear wall buildings", *Proc. of the 11th Conf. on Earthquake Eng.*, Victoria, July 2015.

MOEHLE, Jack, BOZORGNIA, Yousef, and YANG, Tony, "The Tall Building Initiative", *SEAOC convention proceedings*, pp. 315-324, 2007.

OPENSEES, Open System for Earthquake Engineering Simulation (OpenSees), Computer Program and Supporting Documentation, Pacific Engineering Research Centre, University of California, Berkeley, May 2008.

PANAGIOTOU, Marios, "Seismic Design, Testing, and Analysis of Reinforced Concrete Wall Buildings", *Ph.D. Thesis*, University of California, San Diego, 2008.

PANAGOITOU, Marios, and RESTREPO, Jose, "Dual-plastic hinge design concept for reducing higher-rise cantilever wall buildings", *Earthquake Engineering & Structural Dynamics*, Vol. 38, No. 12, October 2009, pp. 1359-1380.

PANNETON, M., LEGER, Pierre., and TREMBLAY, Robert, "Inelastic Analysis of a Reinforced Concrete Shear Wall Building according to the National Building Code of Canada", *Can. J. of Civil Eng.*, Vol. 33, No. 7, July 2006, pp. 854-871.