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# Assessing seismic displacements and ductility demand of tall reinforced concrete shear walls located in eastern Canada

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**ABSTRACT:**

In the force-based and displacement-based methods for seismic design of RC shear walls, geometrical relationship between the base curvature and top displacement is commonly determined considering the displacement profile corresponding to the first-mode deformed shape. Such approach seems appropriate for shorter shear walls, however for taller walls, for which the effects of the higher modes are significant, a more representative relationship needs to be established. In this study, several RC shear walls, with varying heights, lengths, reinforcement details and axial loads are designed for Montreal (QC, Canada) using direct displacement-based design (DDBD) as well as the current Canadian force-based design (FBD) approach. The seismic response of the walls is studied using non-linear time history analysis for selected simulated ground motions compatible with the NBCC design spectrum. The model is developed through the OpenSees platform and calibrated using recent experimental results. Materials and geometric nonlinearities are considered. The relation between the top displacement and base curvature is evaluated from the results of NTHA. The local and global ductility of the shear walls are compared to the estimates obtained by FBD and DDBD approaches.

## 1 INTRODUCTION

Performance-based procedures, and in particular displacement-based design (DBD) methods, continue to gain popularity for seismic assessment of existing structures and design of new structures. Because these procedures use displacements as explicit criteria, the more accurate estimation of the displacement profiles and the relationship between elastic and inelastic displacements is essential. The improvements of displacement estimates are also needed for current codified forced-based seismic design (FBD) procedures, which commonly apply the equal displacement principle to calculate total displacements from elastic displacements. This simplified approach to determine displacements is one of the major inherent shortcomings of the FBD method.

For ductile and moderately ductile reinforced-concrete (RC) shear walls, Canadian concrete design standard A23.3-09 (CSA 2014) requires that verification of rotational deformation demand in the plastic hinge region be conducted. This is usually done by considering that the first mode dominates the displacement profile of the shear wall. While this assumption is appropriate for low-rise structures, for taller walls, higher modes can have significant impact on seismic response and such a simple hypothesis can yield inaccurate predictions. This effect is even more pronounced for structures located in the seismic zones characterised by ground motions rich in high frequencies with greater potential to excite higher dynamic modes of vibration, such as Eastern Canada. The inclusion of higher mode effects to predict displacement profiles is a common issue with force-based and displacement-based methods (Priestley and Kowalsky 2000).

Fundamental mode response is also presumed to estimate the yield point displacement and corresponding rotation and curvature, and establish the relationship between local and global ductility of the RC shear wall structures. For instance, it is commonly assumed that the maximum curvature and maximum roof displacement happen at the same instant. White and Adebar (2004) have demonstrated that such assumption is not very accurate for taller RC shear walls. The intensity of the inelastic response and the length of the plastic hinge can have a considerable impact on the relation between the base rotation and the roof displacement. The relationships available in current design codes for shear walls are generally developed assuming that a large inelastic seismic response takes place. Some recent studies show that, in Eastern Canada, the inelastic response is not significant and is largely overestimated by the current FBD design method (Sadeghian and Koboevic 2013, Luu et al. 2013).

This paper presents a study of seismic deformation profiles of the taller rectangular reinforced concrete shear walls located in Eastern Canada. 15- and 25-storey shear wall buildings are designed following direct displacement design method and current Canadian design provisions. Proposed design spectra and higher mode amplification factors for the upcoming NBCC2015 code (NRC 2015) are also considered in design. Eight design cases are studied with various wall height-to-length and axial load ratios. Nonlinear time history analyses are carried out in OpenSees for a set of ground motions compatible with design spectra to validate the displacement predictions for the two design approaches.

## 2 ESTIMATION OF SEISMIC DISPLACEMENTS FOR TALL SHEAR WALLS

### 2.1 Force-based design approach

In the NBCC force-based methodology for seismic design, displacements are verified at the end of design process, usually after the sections have been sized following the preliminary capacity requirements. The anticipated inelastic response is quantified by the load reduction factors, $R_d$, that are specified for various structural systems and that are directly related to the global ductility factors, $\mu$, by applying the equal displacement principle ($\mu = R_d$). The inelastic seismic base shear is obtained by reducing the elastic seismic base shear with $R_d$. For design, inelastic seismic force is further reduced by the overstrength factor, $R_o$. Once the seismic response of the structure is determined for the reduced level of force, the total displacements of the inelastic system are estimated simply by multiplying the resulting displacement by $R_o*R_d$.

Assuming a predominant first-mode response, the roof displacement and the base curvature of RC shear walls can be related through the following equation (Adebar et al. 2005):

$$\Delta_{in} = \Delta_u - \Delta_y = \left(\varphi_u - \varphi_y\right) * L_{ph} * \left(H_w - 0.5 * L_{ph}\right), \tag{1}$$

where $\Delta_{in}$ is the inelastic part of the total displacement. $\varphi_u$ and $\Delta_u$ are respectively the ultimate base curvature and the ultimate roof displacement. $\varphi_y$ and $\Delta_y$ are respectively the base yield curvature and roof yield displacement. $L_{ph}$ is the length of the plastic hinge and $H_w$ is the height of the wall. In this equation, the ultimate roof displacement, $\Delta_u$ is taken equal to $\mu * \Delta_y$.
From the Eq.1, the following expression can be derived to relate the global displacement ductility, $\mu$, to the local curvature ductility, $\mu_\varphi$:

$$\mu_\varphi = (\mu - 1) * \frac{\Delta_y}{L_{ph}*(H_w - 0.5*L_{ph})*\varphi_y} + 1 \tag{2}$$

The yield curvature is a section property and for the wall section at the base, it can be calculated from equation (3). In this equation $L_w$ represents the length of the wall and $\epsilon_y$ is the yield strain of the steel reinforcement.

$$\varphi_y = \frac{2*\epsilon_y}{L_w} \tag{3}$$

The roof yield displacement is dependent on the distribution of the seismic forces and level of the cracking along the height of the wall. In general, the roof yield displacement can be expressed as:

$$\Delta_y = C_d * \varphi_y * H_w^2 \tag{4}$$

If a uniform cracking distribution is assumed, the coefficient $C_d$ can be taken equal to 0.275 and 0.25 for reverse triangle force distribution and rectangular force distribution, respectively. In the FBD approach, the global ductility of the system, μ, is specified at the beginning of the design process. Therefore, the relation between base curvature and roof displacement is only dependent on height-to-length wall ratio, $H_w/L_w$, and the distribution of the seismic force ($C_d$).

This procedure to relate global and local ductility indicators is based on the assumption thatyield and ultimate deformed shapes of the structure are dominated by the first mode response (Fig.1-a). In this case, the maximum curvature and maximum top displacements occur at the same instant during the earthquake event. For taller RC shear walls located in Eastern Canada this assumption may not be appropriate because the typical high-frequency ground motions in this region are likely to excite higher modes which in turn may significantly affect the seismic response of these structures. As a result, the value of the ultimate roof displacement and the ultimate base curvature could be smaller than the values predicted in design and will unlikely occur simultaneously (Fig. 1-b).

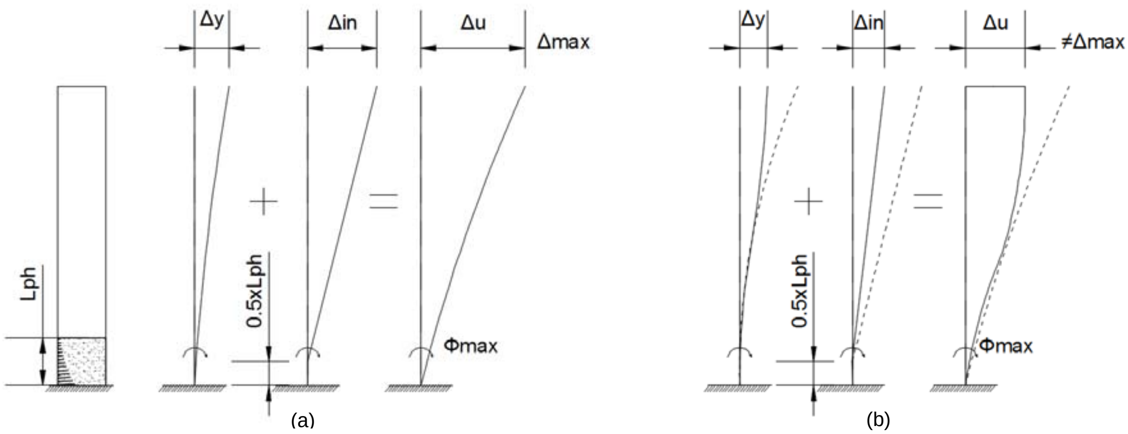


**Figure 1. Seismic deformation profile of rectangular shear walls: (a) First mode dominated displacement profile, (b) The effects of higher modes on displacement profile of tall shear walls.**

## 2.2 Displacement-based design approach

In DBD methods, the design starts with a limitation imposed on displacements and design forces, and ductility corresponding to that displacement are determined consequently. It is crucial to define appropriately the demand displacement profile in order to initiate design process. For a given performance objective, the initial displacement profile for multiple degree of freedom (MDOF) structures is usually estimated from the first-mode deformed shape. The MDOF system is then transformed into an equivalent single degree of freedom system. Using the design displacement determined for the equivalent system and the displacement spectra for design location, the effective period of the structure can be determined and consequently the effective stiffness is derived. Finally, the design base shear is calculated by multiplying the effective stiffness by the design displacement.
In the context of Canadian design norms, the design displacement for the collapse-prevention performance objective could be estimated using two criteria. On one hand, the roof displacement of the shear wall can be taken equal to the maximum allowed inter-story drift index as defined by NBCC ( $\theta_m$ = 2.5%). Once the ultimate roof displacement has been determined, the displacement profile can be derived from the following equations:

$$\Delta_i = \Delta_{yi} + \left(\theta_m - \theta_y\right) * H_i \tag{5}$$

$$\Delta_i = \varphi_y * H_i^2\left(1 - \frac{(1-C_d)*H_i}{H_w}\right) + \left(\theta_m - \frac{C_d*\varphi_y*H_w^2}{H_w - 0.5L_{ph}}\right) * (H_i - 0.5L_{ph}) \tag{6}$$

in which, for a given storey i, $\Delta_i$ is the total displacement, $\Delta_{yi}$ is the displacement at the yield point, $H_i$ is the height of the floor. $\theta_m$ is the maximum allowed drift as per NBCC and $\theta_y$ is the roof drift at the yield point.

On the other hand, design displacement can be calculated as the roof displacement that corresponds to the maximum allowable curvature at the base of the wall, $\varphi_m$.

$$\varphi_m = \frac{\epsilon_{cu}}{c} = \frac{\epsilon_{cu}}{\alpha L_w} \tag{7}$$

where $\epsilon_{cu}$ is the design crushing strain of concrete, equal to 0.0035 in CSA A23.3 14 standard. This requirement guarantees that the inelastic rotation of the shear wall at the base does not exceed the rotational capacity of the section. Adebar et al. (2005) suggest to limit the normalised compression depth, $= c/L_w$ , to 0.40 in order to provide a sufficient rotational capacity for the moderately ductile shear walls with lower ductility demands. Once the maximum allowable base rotation is known, the displacement profile can be obtained from the following expression:

$$\Delta_i = \Delta_{yi} + \Delta_{ini} = \varphi_y * H_i^2 \left(1 - \frac{(1-C_d)*H_i}{H_w}\right) + (\varphi_m - \varphi_y) * L_{ph} H_i \quad (8)$$

The final design displacement profile of the MDOF system is established by selecting the smallest displacement obtained from the two aforementioned criteria for each storey. This deformation profile is then used to calculate the demand shear and bending moments along the height of the shear wall.

## 3 OTHER DIFFERENCES BETWEEN FBD AND DBD APPROACHES TO SEISMIC DESIGN OF TALL RC SHEAR WALLS

In addition to distinct approaches to estimate displacement profiles, the following differences between the two methods can affect design of tall shear walls:

(i) For seismic base shear calculations, NBCC imposes the limits on the fundamental period as a function of the empirical values proposed in the code. Although this restriction is not considered in the estimate of the displacements, larger base shear will affect the final wall dimensions. Because the period limitation is not considered in the DBD approach, tall walls designed by this method will be designed for smaller base shear and will be consequently in general more flexible.

(ii) To guarantee the minimum stiffness and the capacity of the building structure, NBCC prescribes the minimum design base shear force for buildings with fundamental periods longer than 4s which significantly increases force demand for tall shear walls. In the DBD approach, the minimum stiffness required to avoid destructive P-Delta effects is controlled through displacement parameters and this, in general, leads to more rational force demand.

(iii) in NBCC FBD approach, the inelastic design base shear is obtained by reducing the elastic seismic shear by the force reduction factors related to the system ductility ($R_d$) and overstrength ($R_o$). Generally, $R_d$ factors are not explicitly verified in design. It is usually assumed that the proper detailing of the plastic hinge provides anticipated local ductility which in turn results in a design global ductility quantified by $R_d$ factors. Once the design is complete, it is not possible to establish if the level of ductility foreseen in the design is achieved in the structure. For tall shear walls in zones of moderate seismicity, such as Eastern Canada, a predominantly elastic seismic response is likely to occur. This type of response cannot be predicted by FBD methods, so the level of detailing provided in plastic hinge region will increase unnecessarily the construction cost. In DBD approach, ductility of the system is directly calculated during the design process and the response of the structure can be estimated more accurately because the displacement profile of the system at the ultimate and yield point limit states are available. For tall shear walls with large fundamental periods, the maximum wall displacement at the roof will approach the maximum ground displacement and will generally not exceed the maximum design spectral displacement. In regions of moderate seismicity like Eastern Canada, the maximum spectral displacement can be smaller than the predicted yield displacement of the roof. In such cases, the predominantly elastic response of the wall will be correctly predicted by DBD method.

## 4 NUMERICAL STUDY

### 4.1 Description of the design

To study the deformation response, eight shear walls were designed for Montreal, Quebec, following FBD and DBD methods. The FBD design was carried out in accordance with NBCC 2010 and the CSA A23.3 14 provisions for moderate ductile RC shear walls ($R_d$ = 2 and $R_D$ = 1.4). Displacement-based design was carried out using direct displacement-based design (DDBD) procedure proposed by Priestly and Kowalsky (2000). The application of this method is explained in Sadeghian and Koboevic (2013). For two methods, the acceleration design spectrum and higher modes amplification factors proposed for the upcoming 2015 edition of NBCC were used. The displacement design spectrum was constructed from acceleration design spectrum in the absence of explicitly defined code values.

The selected shear walls provided the lateral resisting system for a regular office building with rectangular floor plan. Two building heights were considered: 15-storeys (44.75m) and 25-storeys

(74.25m). Response spectrum analysis was used to determine the effects of seismic loads. Accidental torsion was considered in the analysis with the two methods. In all cases, the minimum wall length that satisfied design criteria for selected design approach was chosen. For each wall height, two levels of vertical axial forces were considered; $P1 = 0.08*f_c'*A_c$ and $P2= 0.12*f_c'*A_c$, where $A_c$ is the gross area of the wall cross section, and $f_c'$ is the compressive strength of concrete taken equal to 30 MPa. The summary of the FBD and DDBD designs is shown in Table 1.

For walls designed using the FBD approach, the empirical limit on the fundamental period lead to much larger design accelerations and consequently higher design forces. The bending moments $M_f$, and the shear forces $V_f$ at the base of the walls, shown in Table 1, are determined using the capacity design procedure by assuming full formation of the plastic hinge. Larger axial force, P2, increased the bending moment and shear capacity of the wall at the base, but it reduced the inelastic rotation capacity of the plastic hinge. For this reason, the wall length selected for each building height was identical in spite of the different level of axial load (see column $L_w$ in Table 1).

**Table 1. Summary of wall designs**

| **Design case** | **$H_w$ (m)** | **$L_w$ (m)** | **$t_w$ (mm)** | **$T_1$ (s)** | **$V_d$ (kN)** | **$M_d$ (kNm)** | **$\Delta$ (mm)** |
|---|---|---|---|---|---|---|---|
| FBD-15-P1 | 44.75 | 7.00 | 350 | 4.04 | 5990 | 49154 | 145 |
| FBD-15-P2 | 44.75 | 7.00 | 350 | 4.04 | 5990 | 49154 | 145 |
| FBD-25-P1 | 74.25 | 9.00 | 450 | 6.53 | 8489 | 96094 | 520 |
| FBD-25-P2 | 74.25 | 9.00 | 450 | 6.53 | 8489 | 96094 | 520 |
| DDBD-15-P1 | 44.75 | 6.00 | 400 | 4.97 | 1004 | 14656 | 177 |
| DDBD-15-P2 | 44.75 | 6.00 | 400 | 4.97 | 1119 | 16294 | 177 |
| DDBD-25-P1 | 74.25 | 8.00 | 500 | 7.78 | 894 | 20407 | 230 |
| DDBD-25-P2 | 74.25 | 8.00 | 500 | 7.78 | 1007 | 22948 | 230 |

$H_w$: wall height, $L_w$: wall length, $t_w$: wall thickness, $T_1$: fundamental period, $\Delta$: top displacement

For all DDBD cases, the maximum spectral displacement (154mm) was smaller than the yield displacement which indicates that the response of the system will be predominantly elastic and can be characterised by the ductility factor equal to one. As no formation of plastic hinge was anticipated, no particular ductile detailing was implemented for these walls. The minimum length of the walls needed to control P-$\Delta$ effects was selected by limiting the stability index $SI = (P * \Delta)/(V * h)$ to 0.4. As the design base shear and bending moments for DDBD cases were much smaller than their FBD counterparts and the global stability criterion was critical for design, the final overstrength factors at the base of these walls were large (2.2 to 3.6) and exceeded 1.3 obtained for FBD walls.

P-$\Delta$ effects in FBD method were considered negligible ($SI < 0.1$) so the axial load had no impact on design base shear contrary to DDBD approach for which the vertical axial load had direct effect on the effective stiffness and thereby on the seismic design base shear. The top displacements for 25-storey FBD cases shown in Table 1 exceed significantly those obtained for DDBD walls which were equal to the maximum spectral displacement. Such large values of top displacement are not realistic and can be attributed to the fact that the global ductility that was foreseen in FBD design and used to calculate displacements did not in fact fully develop.

### 4.2 Non-linear time history analysis

The response of the shear walls was studied using nonlinear time history analysis on OpenSees platform (PEER 2000). The walls were modeled using one force-based nonlinear beam-column element per floor. The section of the element was discretized into fibers for which the nonlinear material stress-strain response was defined. Distinct fibers were defined for confined and unconfined concrete zones and for the steel reinforcement. The fiber section model considers the bending moment and axial load interaction, but the shear-bending or shear-axial load interactions cannot be represented.

Concrete behaviour was modeled using the uniaxial Kent–Scott–Park model with linear tension softening (Concrete02). To determine the material parameters, the material law based on modified compression field theory proposed by Vecchio et al. (1986, 2000) was used. For unconfined concrete, the ultimate strain, $\varepsilon_{cu}$, was taken equal to 0.0035 as prescribed by CSA A23.3, and the ultimate concrete compressive strength was assumed to be 30 MPa. For reinforcing bars, the Giuffré-Menegotto-Pinto hysteretic material (Steel02) was employed to describe the inelastic behaviour.

The parametric study was conducted to determine the adequate number of integration points for the analysis. After examining the structural periods, wall base shear and storey shears, overturning moments and roof displacements, three integration points per element were selected as a rational compromise offering adequate accuracy, convergence and reasonable computational time. This is in line with the practice reported in the literature (Boivin et al, 2012).

5% damping for first and third mode was assigned and the initial stiffness option was used to constitute the damping matrix. For $W_{MTL}$ models, for which the lesser inelastic response was anticipated, the strain hardening was taken equal to 1.2% as suggested by Ghorbanirenani (2010) and for $W_{VRC}$ models with expected high inelastic response the strain hardening was taken equal to 2% as discussed in ATC72-1(2010). The models were calibrated on the basis of available experimential data for rectangular shear walls located in Eastern Canada (Ghorbanianrenani et al. 2012) following the recommendations by Luu et al. (2013). The analysis were conducted for a suite of seven simulated ground motions, five characterized by low-frequency content, and two with high-frequency content. The records were selected on basis of predominant M-R scenarios and scaled to match the NBCC 2015 acceleration design spectrum.

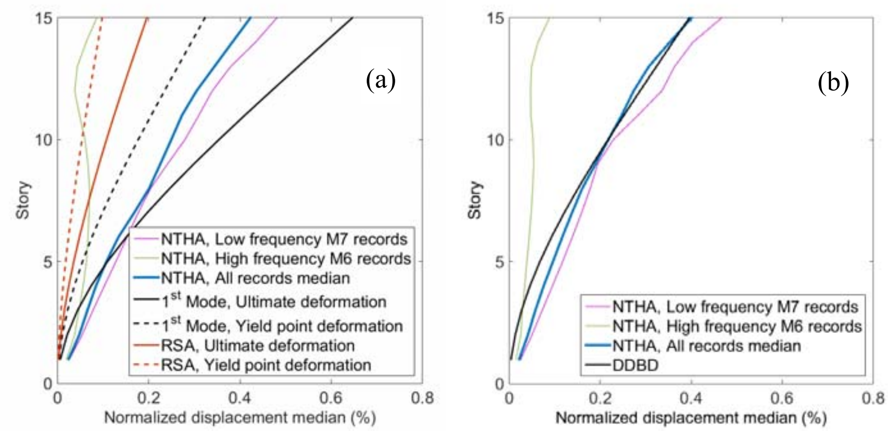


**Figure 2. Normalized displacement profile for 15-storey shear walls: (a) FBD-15-P2 design case, and (b) DDBD-15-P2 design case**

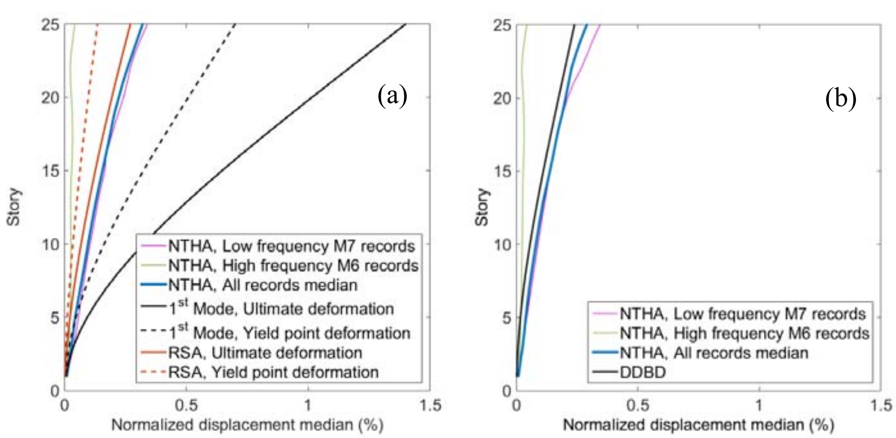


**Figure 3. Normalized displacement profile for 25-storey shear walls: (a) FBD-25-P2 design case, (b) DDBD-25-P2 design case**

The displacements from NTHA, normalised by the height of the wall, are shown in Figs. 2 and 3 for 15- and 25-storey walls, respectively. FBD cases are shown on the left (a) and DBDD cases on the right (b). The axial load had a negligible impact on deformation profiles and thus only the results for P2 load are illustrated. The median values of maximum displacements are given for the complete set of records and for low- and high-frequency records separately. Predicted displacement profiles from FB and DDB designs are added for comparison. FBD results include the predicted yield and ultimate displacements obtained from complete response spectrum analysis (RSA) as well as considering the contribution of the first mode only ($1^{st}$ mode). For DDBD cases, the deformations shown are elastic and based on the first mode response, consistently with the type of response predicted by this method and its fundamental assumptions.

The curvatures from the NLTH analysis, normalised by the yield curvature of the wall section that was determined according to Eq. 3, are illustrated in Figs. 4 (a) and (b) for the 15- and 25-storey walls respectively. The graphs show the median values of the maximum curvatures recorded at a given level for each ground motion record. Results are presented both for P1 and P2 axial loads because the level of axial load had impact on curvature.

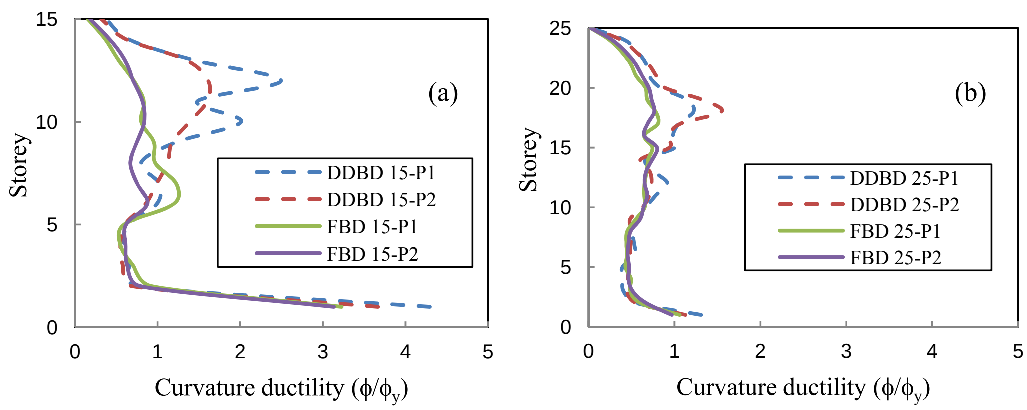


**Figure 4. Curvature profile: (a) 15-storey walls (b) 25-storey walls**

## 5 DISCUSSION OF THE RESULTS

Results of NTHA show that the FBD and DDBD methods lack accuracy in estimating the displacement profile and the base curvature. For the 15-storey FBD wall, the median displacements are smaller than the predicted ultimate displacements based on the first mode but they significantly surpass yield and ultimate displacements predicted by RSA. The median results from the complete set of records match the median obtained for the low-frequency records, with the top median drifts reaching 0.4% and 0.5%, respectively. As expected, the displacements caused by the high-frequency records were significantly smaller. Conversely, for the 25-storey FBD cases, the NTHA results are very close to the ultimate displacement predicted by RSA, but much smaller than the predicted yield and ultimate displacements based on the first mode profile, reaching the median top drift value equal to 0.3%. The median value of the top displacement for 15-storey DDBD obtained from the NTHA was 0.4% and matched well the value predicted by design. However, up to the mid-height of the wall, the difference between the NTHA results and the predicted displacements are more significant. The difference can be attributed to intrinsic problems related to the fiber-section modelling which does not allow adequate representation of the development of the plastic hinge over the height of the wall. For this reason, the inelastic rotation causes larger drifts and displacement in the lower stories compared to the design predictions. This phenomenon was observed for both design methods. The predicted top displacement for the 25- storey DDBD wall, equal to 0.3%, is slightly below the NTHA results. Note that, in spite of the difference in length between the FBD and DDBD walls, for a given building height, the top displacements are similar.

In FDB procedure, walls were designed as moderately ductile considering the ductility-related force reduction factor, $R_d = 2$ . $R_d$ corresponds to global system ductility, . For DDBD designs, the global system ductility is equal to one as the elastic response was predicted. The predicted base curvature ductility, $\mu_\varphi$ (local ductility) was determined according to Eq. 2 and shown in Table 2 for both methods. For 15- and 25- storey FBD walls $\mu_\varphi$ is equal to 2.69 and 2.86 respectively, while it takes value of one for the DDBD designs. Fig. 4 shows that the median base curvature for 15-storey shear wall is equal to 3 and only slightly exceeds the FBD predicted value. However, the discrepancies are much more pronounced for DDBD design cases for which median base curvature of 3.7 and 4.3 were recorded. For 25-storey shear walls, for which the elastic response was predicted by DDBD method, a good match between design predictions and NLTH results was observed. On the other hand, FBD method overestimated the local ductility demand at the base by about two times. Curvature profiles obtained for more slender DDBD walls indicate the possibility of the formation of a second plastic hinge at around two-third of the wall height for both building heights studied. Such tendency was not observed for less slender FBD walls.

The values of the curvature ductility at the base of 25-storey wall show that limited inelastic response is expected. Because their fundamental periods are very large, it is anticipated that the top displacements will be smaller than the maximum spectral displacement (154mm). However, Table 2 shows that for both design methods, the top displacements are approximately equal to two times the maximum spectral displacement. Because of the height of the tall shear walls, a minor inelastic rotation at the plastic hinge could cause significant top displacement. This sensitivity is even more emphasised when analytical models cannot adequately represent the length of the plastic hinge.

**Table 2. Comparison of the base curvature ductility predictions**

| Design case | NTHA | | | Design |
|---|---|---|---|---|
| | Base curvature | Top displacement (mm) | $\mu_\varphi$ | $\mu_\varphi$ |
| FBD-15-P1 | 0.001844 | 292 | 3.2 | 2.69 |
| FBD-15-P2 | 0.001791 | 246 | 3.1 | 2.69 |
| FBD-25-P1 | 0.000471 | 303 | 1.1 | 2.86 |
| FBD-25-P2 | 0.000431 | 309 | 1.0 | 2.86 |
| DDBD-15-P1 | 0.002863 | 242 | 4.3 | 1 |
| DDBD-15-P2 | 0.002445 | 233 | 3.7 | 1 |
| DDBD-25-P1 | 0.000654 | 294 | 1.3 | 1 |
| DDBD-25-P2 | 0.000564 | 279 | 1.1 | 1 |

## 6 CONCLUSIONS

A study of seismic deformation profiles of the taller rectangular RC shear walls located in eastern Canada was carried out for the 15- and 25-storey shear wall buildings designed using DDBD and FBD current Canadian design provisions. Following conclusions were drawn:

- The two methods show inaccuracy in estimating the deformation response parameters including the base curvature, ductility and top displacement. However, the DBDD approach provided better estimation of the top displacement for both building heights under study.
- The FBD method does not give a consistent prediction of the top displacement. A significant difference was observed between the results based on the fundamental modal shape and the results obtained by the complete response spectrum analysis. The response spectrum analysis does not necessarily lead to a better estimation of the displacements and for shorter (15-storey) shear walls it can even significantly underestimated the top displacement.
- The inelastic response of the shorter walls is much more pronounced compared with the taller ones. This is in line with the predictions of the DDBD approach.
- DDBD approach provides satisfactory estimates of curvature ductility for taller shear walls, but it can underestimate the level of inelastic response for the shorter walls.
- The vertical axial load does not have notable effect on the deformation response of the wall.
- Walls designed using DDBD approach have in general smaller lengths and are more slender. However, this slenderness increases the chance of formation of the second plastic hinge at two-third of the height of the walls.

- For taller walls the top displacement is very sensitive to changes at the base curvature. Small inelastic response at the base could largely amplify the top displacement.

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