



## **DISPLACEMENT-BASED DESIGN OF CONCRETE WALL BUILDINGS: THE 2004 CANADIAN CODE PROVISIONS**

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### **SUMMARY**

This paper presents the new displacement-based provisions for the seismic design of concrete wall buildings in the 2004 Canadian code. According to these provisions, an estimate of the lateral displacement of the building due to the design-basis earthquake is required in order to evaluate the flexural ductility (confinement requirements) of concrete walls, shear design requirements in plastic hinge regions of flexural walls, punching shear stress limits for slab-column connections in frames that are not part of the seismic force resisting system, and seismic design requirements for tilt-up concrete walls with large openings.

### **INTRODUCTION**

In the seismically active region of western Canada, concrete walls are the preferred seismic force resisting system for all types of buildings from the smallest industrial/commercial building with tilt-up walls to the tallest high-rise building with coupled walls. Unlike US building codes, Canadian building codes do not place a limit on the maximum height of buildings with ductile concrete walls.

The design requirements for earthquake loads and effects are given in the National Building Code of Canada (NBCC). The 2005 edition of NBCC has been extensively revised from the current edition. The design-basis earthquake has been increased to the maximum considered earthquake with a 2% in 50-year probability of exceedance (2500 yr return period). To compensate for the increased seismic forces, the overstrength of structures is now explicitly accounted for using overstrength force reduction factors.

The specific design and detailing requirements for concrete systems are given in Clause 21 of the Canadian concrete code (CSA Standard A23.3 – Design of Concrete Structures). A number of the provisions in that code for the seismic design of concrete wall buildings have been completely revised for the 2004 edition. Many of these new provisions are displacement-based, and thus require an estimate of the lateral displacement of the building due to the design-basis earthquake. This paper presents the new displacement-based provisions for concrete wall buildings.

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### Concrete Wall Systems in 2005 NBCC

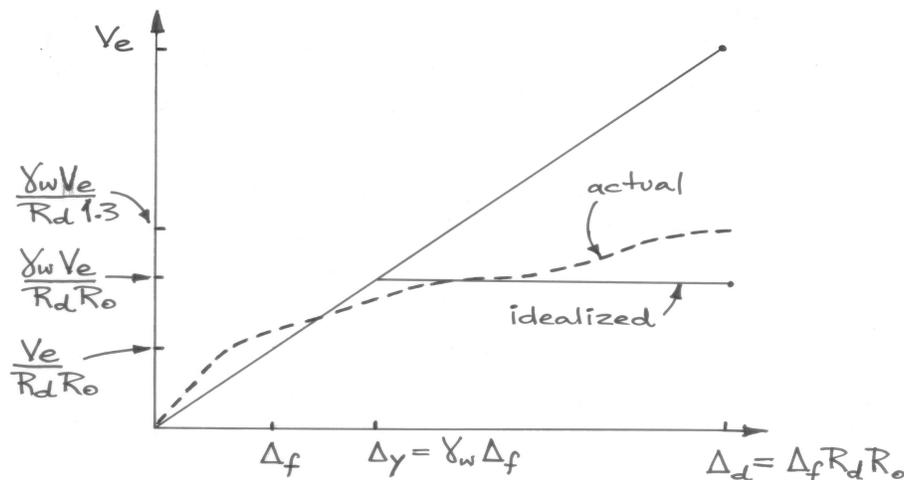
Table 1 summarizes the concrete wall systems defined in the 2005 National Building Code of Canada and the corresponding seismic force reduction factors. The 2005 NBCC has two force reduction factors: a ductility force modification factor  $R_d$ , and an overstrength force modification factor  $R_o$ . The ductility force modification factor is similar to the force reduction factor in the current NBCC. The overstrength force modification factor is made up of three parts (Mitchell et al. [1]): (1) 1.3 for all wall systems to account for the difference between actual and factored strength; about two-thirds of this 30% increase is due to the difference between specified and factored strength, while the remainder is due to the difference between actual and specified. (2) A variable factor to account for the increased stresses in reinforcement at high tensile strain levels due to strain hardening of reinforcement; 1.25 for ductile walls, 1.1 for moderately ductile walls, and 1.0 for conventional walls. (3) A factor of 1.05 to account for the increased forces required to develop a complete mechanism in coupled and partially coupled walls.

**Table 1: Concrete Wall Seismic Force Resisting Systems (SFRS) in the 2005 NBCC.**

Type of SFRS	$R_d$	$R_o$	$R_d \times R_o$	Limitation
Ductile Coupled Walls	4.0	1.7	6.8	-
Ductile Partially Coupled Walls	3.5	1.7	6.0	-
Ductile Shear Wall	3.5	1.6	5.6	-
Moderately Ductile Shear Wall	2.0	1.4	2.8	60 m height*
Conventional Shear Wall	1.5	1.3	2.0	30 m height*

\* Except in low seismic regions

The linear (dynamic) analysis of a building is normally done using the factored (reduced) force  $V_e/(R_d R_o)$ , where  $V_e$  is the elastic base shear. As shown in Fig. 1,  $\Delta_f$  is the corresponding factored displacement of the structure, and  $\Delta_f R_d R_o$  is the total displacement demand corresponding to the elastic base shear  $V_e$ . Throughout the 2004 Canadian concrete code (and this paper),  $\Delta_f R_d R_o$  is used to refer to the total displacement demand.



**Fig. 1 – Idealized load-deflection response in terms of 2005 NBCC terminology.**

In determining the inelastic portion of the displacement demand on concrete wall systems, the actual yield strength of the system is needed. This is done using the wall overstrength factor  $\gamma_w$ , which is equal to the ratio of actual yield strength to factored demand on the wall. To be consistent with the derivation of the overstrength force modification factor  $R_o$ , the overstrength factor  $\gamma_w$  should be taken as 1.3 times the ratio of factored strength to factored demand. That is, if a wall has a factored strength equal to 10% higher than the factored demand, the overstrength  $\gamma_w$  is equal to  $1.3 \times 1.10 = 1.43$ . Thus, the actual yield strength of a structure can be expressed as  $\gamma_w V_e / (R_d R_o)$  as shown in Fig. 1.

## EFFECTIVE STIFFNESS OF WALLS

The displacement demand of a structure depends on, among other things, the stiffness of the structure. For concrete structures, the effect of cracking must be accounted for, and in linear analysis, this is usually done using simple reduction factors on gross (uncracked concrete) section properties. For example, the flexural rigidity  $E_c I_e$  is normally taken as  $\alpha E_c I_g$ , where one value of  $\alpha$  is used for an entire structure.

Adebar and Ibrahim [2] investigated the effect of cracking on flexural stiffness of concrete walls, and found that the response during any load cycle depends on the maximum deformation amplitude in previous cycles. To simplify the problem, they recommended the use of an upper and lower bound response corresponding to a previously uncracked wall and a severely cracked wall, respectively. Ibrahim and Adebar [3] presented simple expressions for upper-bound and lower-bound flexural rigidity of concrete walls determined from the slope of the elastic portion of an equivalent elastic-plastic load-deflection curve that has the same area under-the-curve as the actual nonlinear relationship. The level of axial compression at the base of the wall was found to have the greatest influence on the average flexural rigidity of the entire wall. The amount of vertical reinforcement, concrete strength, yield strength of vertical reinforcement, and shape of cross section were found to have less influence. The following expressions were proposed for the reduction factor on flexural rigidity  $\alpha = E_c I_e / E_c I_g$ :

*Upper-bound:*

$$(1) \quad \alpha = 0.6 + \frac{P}{f_c' A_g} \leq 1.0$$

*Lower-bound:*

$$(2) \quad \alpha = 0.2 + 2.5 \frac{P}{f_c' A_g} \leq 0.7$$

where  $P/A_g$  is the axial compression stress at the base of the wall due to gravity loads. For multiple wall segments, a single reduction factor based on an average value of  $P/A_g$  may be used to estimate the overall stiffness of the structure.

For any given wall, the effective stiffness will be somewhere between these two bounds. In the initial draft of the 2004 Canadian concrete code, the upper-bound reduction factor was proposed for moderately ductile ( $R_d = 2.0$ ) walls and the lower-bound reduction was proposed for ductile ( $R_d = 3.5$ ) walls. The rationale was that walls designed with a larger force reduction factor would be more likely to be severely cracked during an earthquake. The 2005 NBCC overstrength force reduction factors (shown in Table 1) maintain the design force levels similar to historical levels even though the design earthquake has been significantly increased; however these factors have no compensating effect on the design displacements (see Fig. 1). Due to concerns that the combined effect of larger design-basis earthquakes and drastically

reduced effective stiffnesses would have too much of an effect on final designs, it was decided to use the upper-bound effective stiffness for all concrete walls until such time as better information is available on the choice of effective stiffness between the upper and lower bounds.

The flexural stiffness of a coupled wall system depends on the axial stiffness of the walls and the shear and flexural stiffness of the coupling beams. The axial stiffnesses of the walls need to be reduced to account for cracking in the same way as the flexural stiffness of a cantilever wall is reduced. That is, the effective cross sectional area  $A_e = \alpha A_g$ , where  $\alpha$  is given by Eq. (1). The axial stress  $P/A_g$  used to calculate  $\alpha$  is due to the gravity loads only. The effect of coupling beam forces can be ignored for simplicity because the coupling beam forces reduce the compression in one wall and increase the compression in another wall, but do not change the average compression on all walls.

The commentary to the current Canadian concrete code [4] recommends reduction factors of 0.4 and 0.2 on the flexural rigidity of coupling beams with diagonal and conventional reinforcement, respectively. A recent full-scale test on a slender diagonally reinforced coupling beam [5] has shown that a reduction factor of 0.4 is not adequate for diagonally reinforced coupling beams. In the 2004 Canadian code, the following new reduction factors are given: 0.45 and 0.15 on the shear stiffness and 0.25 and 0.4 on the flexural rigidity of coupling beams with diagonal and conventional reinforcement, respectively. The combined effect of these shear and flexural reduction factors is similar to the effect of the reduction factors on flexural rigidities in the current New Zealand concrete code [6]. A simple rationale for the relative values of the new reduction factors is as follows: conventional reinforcement (longitudinal and transverse) provides reinforcement where needed to effectively control flexural cracking; but not diagonal (shear) cracking. Diagonal reinforcement provides effective diagonal crack control; but not efficient flexural crack control. The results of a study [5] suggests that the length of coupling beams be taken as 20% longer than the clear span of beams to account for significant strain penetration into adjoining walls.

## DUCTILITY OF FLEXURAL WALLS

The new provisions for when confinement reinforcement is required in walls are displacement-based, but are formulated in terms of inelastic rotations. To ensure that a wall has adequate ductility in the plastic hinge region, the inelastic rotational capacity of the wall  $\theta_{ic}$  must be greater than the inelastic rotational demand  $\theta_{id}$ .

### Inelastic Rotational Capacity

The inelastic rotational capacity of a wall is given by:

$$(3) \quad \theta_{ic} = (\phi_c - \phi_y) l_p$$

where the total curvature capacity  $\phi_c$  equals the maximum compression strain of concrete  $\epsilon_{cm}$  divided by the compression strain depth  $c$ ; the yield curvature  $\phi_y$  can be safely estimated as  $0.004/l_w$  for walls without confinement, and the plastic hinge length  $l_p$  can be safely taken as  $0.5 l_w$ . Substituting these values into Eq. (3) results in the following expression for the inelastic rotational capacity of a concrete wall:

$$(4) \quad \theta_{ic} = \left( \frac{\epsilon_{cm} l_w}{2c} - 0.002 \right)$$

where  $\epsilon_{cm}$  shall be taken as 0.0035 unless the compression region of the wall is confined as a column. While a maximum compression strain of up to 0.004 is appropriate for unconfined concrete, 0.0035 is

used in order to be consistent with the maximum compression strain used for calculating flexural strength. A description of how to determine  $\varepsilon_{cm}$  for walls with confinement is given by Mitchell et al. [7].

When the concrete compression strain demands are small, the displacement capacity of a concrete wall may be limited by the tensile strain capacity of reinforcing steel, which for bonded reinforcing bars embedded in concrete is about 0.05. This strain capacity is much less than the strain capacity of bare reinforcing bars due to the localization of strains at cracks. A conservative curvature capacity of  $0.05/l_w$  results from assuming zero compression strain in concrete, and the corresponding inelastic rotation is  $0.05/l_w \times 0.5 l_w = 0.025$ . Thus the inelastic rotation capacity of a wall must be limited to 0.025. As the 2005 NBCC limits the maximum inter-story drift to 0.025, the inelastic rotational demand will always be less than 0.025 for a concrete wall meeting the inter-story drift requirements of the 2005 NBCC.

To the author's knowledge, there are no recommendations for plastic hinge length of coupled walls. It is expected that walls with a low degree of coupling will act similar to separate cantilever walls. Thus, the wall length to be used in Eq. (4) for estimating the rotational capacity of Partially Coupled Walls (degree of coupling < 67%) is the individual wall segment length. On the other hand, very highly coupled walls will act similar to a single solid wall with openings. In the absence of any better information, it is recommended that the wall length used in Eq. (4) to estimate the rotational capacity of Coupled Walls (degree of coupling  $\geq 67\%$ ) be the overall length of the coupled system. This approach is consistent with the simple approach used to extend the procedures for cantilever walls to coupled walls in the 1984/1994 Canadian concrete codes.

### Inelastic Rotational Demand

The inelastic rotational demand on a concrete wall can be determined from:

$$(5) \quad \theta_{id} = \frac{\Delta_{id}}{(h_w - 0.5l_p)}$$

where  $\Delta_{id}$  is the inelastic displacement demand, and  $(h_w - 0.5l_p)$  is the effective height of the wall above the centre of the plastic hinge. While a shorter plastic hinge length of  $l_p = 0.5l_w$  gives safer results when estimating rotational capacity from Eq. (4), a longer plastic hinge length of  $l_p = 1.0l_w$  gives safer results when estimating rotational demand from Eq. (5). Walls of different length that are tied together at numerous floor levels experience similar plastic rotations if they are subjected to the same top displacement (Adebar et al. [8]). Thus, one value of  $(h_w - 0.5l_p)$  should be used for an entire system of walls acting together, and to be safe,  $l_w$  should be from the longest wall in the system.

The remaining unknown in Eq. (5) is the inelastic displacement demand, which is the difference between the total displacement and the elastic displacement:

$$(6) \quad \Delta_{id} = \Delta_d - \Delta_y$$

One approach is to assume that the elastic portion  $\Delta_y$  is equal to the first mode yield displacement, which is a function of the wall height and length. This approach cannot easily be extended to coupled walls or systems with different length walls, as the yield displacement of these is not related to the dimensions of any individual wall. Another problem is that the first mode yield displacement increases exponentially with wall height; but the displacement demand of very tall walls will be limited by the maximum ground displacement. The solution to both of these is to determine the inelastic displacement as a portion of the total displacement demand of the seismic force resisting system. One method of doing this is to assume that the inelastic drift, which is equal to the inelastic rotation, is equal to the maximum global drift:

$$(7) \quad \theta_{id} = \frac{\Delta_{id}}{(h_w - 0.5l_p)} \approx \frac{\Delta_d}{h_w}$$

This approach was used to develop the wall ductility provisions in the 1999 ACI 318 building code (Wallace and Orakcal [9]). White and Adebar [10] compared the inelastic rotations determined using Eq. (7) with the results of numerous nonlinear dynamic analyses on high-rise buildings, and found that this approach gives reasonable results for coupled walls; but may over predict the inelastic rotations in cantilever walls. The reason is that the elastic displacement  $\Delta_y = \Delta_d - \Delta_{id}$  may be a larger portion of the total displacement than half the wall length ( $0.5l_w$ ) is of the total wall height  $h_w$  in tall buildings.

Another possible approach is to relate the elastic portion of the total displacement to the relative strength of the wall. Unlike Eq. (7), such an approach would predict that a wall with adequate strength would not be subjected to any inelastic displacement demand, and would predict that an increasing portion of the total displacement is due to inelastic displacement as the strength is reduced. A simple rational expression for the inelastic portion of the total displacement demand is given by:

$$(8) \quad \Delta_{id} = \Delta_d \left( 1 - \frac{1}{R} \right)$$

where  $R$  is equal to the ratio of elastic demand to strength of the wall. White and Adebar [10] found good agreement between the predictions from Eq. (8) and the results from non-linear dynamic analyses of a variety of high-rise buildings with cantilever walls subjected to a wide variety of ground motions.

Substituting Eq. (8) into Eq. (5) and introducing the 2005 NBCC terminology given in Fig. 1 results in the following expression for inelastic rotational demand on cantilever walls:

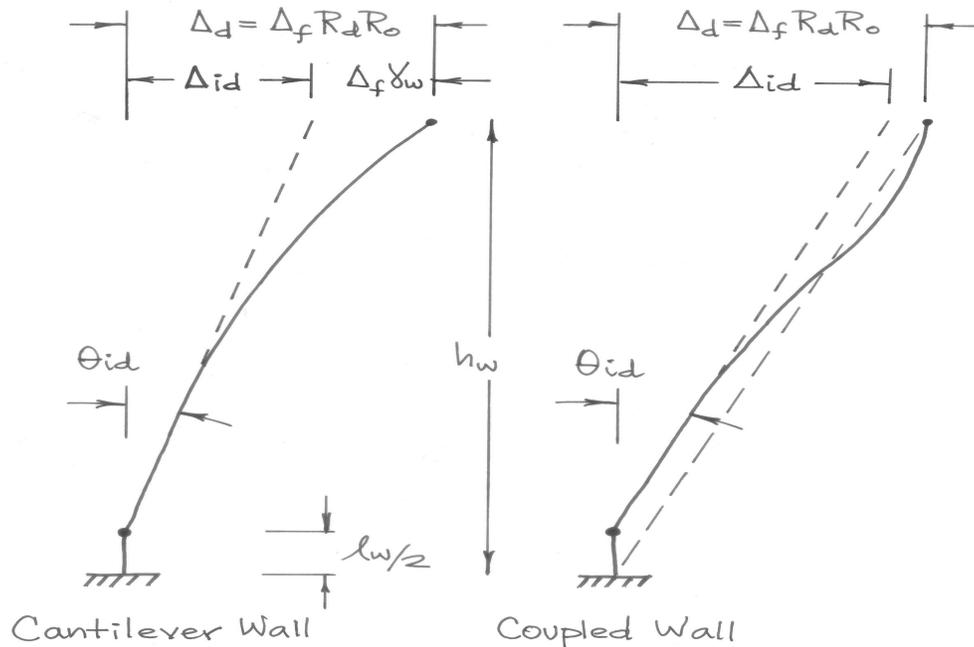
$$(9) \quad \theta_{id} = \frac{\Delta_f (R_d R_o - \gamma_w)}{h_w - 0.5l_w}$$

White and Adebar [10] found that Eq. (8) gives unsafe predictions for buildings with coupled walls, particularly if the building has uniform coupling beam strengths over the height. Introducing the 2005 NBCC terminology given in Fig. 1 into Eq. (7) results in the following expression for inelastic rotational demand on coupled walls:

$$(10) \quad \theta_{id} = \frac{\Delta_f R_d R_o}{h_w}$$

Figure 2 depicts the reason for the difference between Eq. (9) for cantilever walls and Eq. (10) for coupled walls. A significant portion of the total displacement at the top of cantilever walls is due to elastic curvatures, and this is estimated as  $\Delta_f \gamma_w$  in Eq. (9). Due to the “pull-back” from coupling beams, the elastic curvatures result in much less elastic top wall displacement of coupled walls. Thus, while coupling walls together will reduce the total displacement, a larger portion of that total displacement will cause inelastic rotation at the base.

To ensure that all Ductile Walls (coupled, partially coupled or cantilever) have a minimum amount of ductility, the inelastic rotational demand shall not be taken less than 0.004. For Moderately Ductile walls, the inelastic rotational demand shall not be less than 0.003.



**Fig. 2 – Relationship between total displacement demand at top of wall and inelastic rotational demand at base of wall for: (a) cantilever walls, and (b) coupled walls.**

### Simplified Expressions for Wall Ductility

The ductility requirements of the 2004 Canadian concrete code are presented in terms of inelastic rotations to ensure that designers have an appreciation for the inelastic mechanism; however the provisions can be presented in terms of maximum compression depth as a ratio of wall length. Setting the inelastic rotational capacity of unconfined walls given by Eq. (4) (with  $\epsilon_{cm} = 0.0035$ ) to be greater than or equal to the inelastic rotational demand on cantilever walls given by Eq. (9) and rearranging gives:

$$(11) \quad \frac{c}{l_w} \leq \frac{0.875}{1 + 500 \left( \frac{\Delta_f R_d R_o}{h_w} \right) \left[ \frac{1 - \gamma_w / (R_d R_o)}{1 - 0.5l_w / h_w} \right]} \leq 0.3$$

The  $c/l_w$  limit of 0.3 results from the minimum inelastic rotational demand of 0.004 for ductile systems. The corresponding limit for Moderately Ductile Shear Walls is 0.35 due to the minimum inelastic rotation demand being only 0.003. For coupled and partially coupled walls, the term within the square brackets does not exist (should be taken as 1.0).

The maximum compression depth as a ratio of wall length depends primarily on the global drift demand  $\Delta_f R_d R_o / h_w$ . For cantilever walls, the global drift in Eq. (11) is multiplied by a correction factor (within square brackets) that converts global drift into inelastic rotation (drift). The numerator within the correction factor converts total displacement into inelastic displacement, and the denominator within the correction factor converts total height of wall to height above the centre of the plastic hinge.

Eq. (11) can be further simplified by substituting values for the variables within the correction factor (square brackets). The height-to-length ratio of the wall  $h_w/l_w$  does not have a significant influence except when the aspect ratio is below about four; but in that range the basic flexural mechanism that was

assumed in developing the equation becomes questionable. Simply assuming  $h_w/l_w = 5$  is a reasonable approximation for most walls. The overstrength factor  $\gamma_w$ , which is equal to 1.3 times the ratio of factored strength to factored demand, has a minimum value of 1.3. The following expressions are appropriate for each of the three types of concrete walls:

*Ductile Coupled and Partially Coupled Walls:*

$$(12) \quad \frac{c}{l_w} \leq \frac{0.875}{1 + 500 \left( \frac{\Delta_f R_d R_o}{h_w} \right)} \leq 0.3$$

where  $l_w$  is the overall length of the coupled system for fully coupled walls, and the individual wall length for partially coupled walls.

*Ductile Shear Walls:*

$$(13) \quad \frac{c}{l_w} \leq \frac{0.875}{1 + 425 \left( \frac{\Delta_f R_d R_o}{h_w} \right) [1.3 - 0.2\gamma_w]} \leq 0.3$$

*Moderately Ductile Shear Walls:*

$$(14) \quad \frac{c}{l_w} \leq \frac{0.875}{1 + 300 \left( \frac{\Delta_f R_d R_o}{h_w} \right) [1.9 - 0.7\gamma_w]} \leq 0.35$$

Note that the influence of any overstrength beyond the minimum  $\gamma_w$  of 1.3 is particularly significant for Moderately Ductile Shear Walls.

The current ACI 318 building code has the following limit on concrete walls without special boundary elements (confinement):

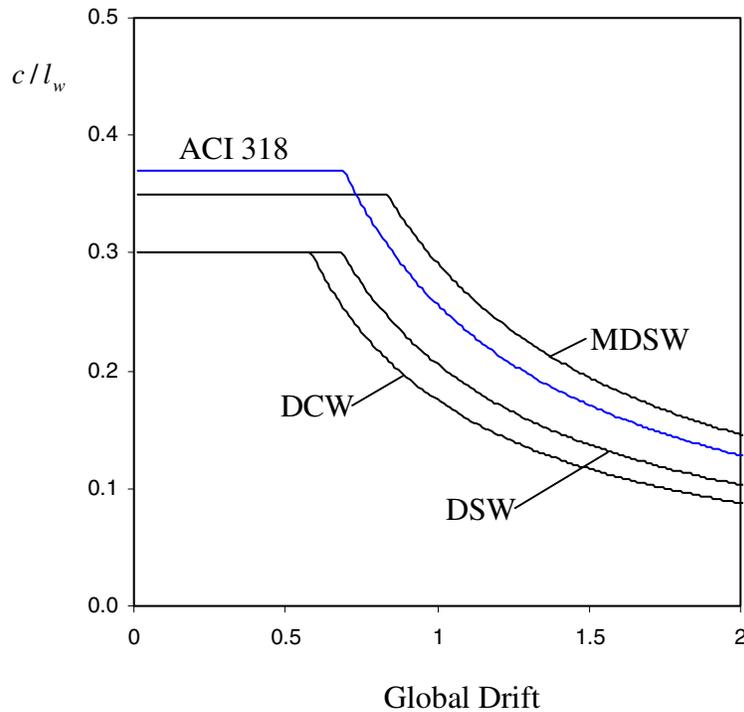
$$(15) \quad \frac{c^*}{l_w} \leq \frac{0.875}{600 \left( \frac{\delta_u}{h_w} \right)} \leq 0.238$$

where  $c^*$  indicates that the compression depth is calculated using the nominal concrete strength  $f'_c$  rather than the factored compressive strength  $\phi_c f'_c = 0.65 f'_c$  used in Canada.  $\delta_u$  is the total lateral displacement expected for the design-basis earthquake, which is equal to  $\Delta_f R_d R_o$  in 2005 NBCC terminology.

Assuming the concrete compression zone has a uniform width so that Eq. (15) can be converted for use with the factored compressive strength of concrete by dividing all terms in Eq. (15) by 0.65, results in the following equivalent expression:

$$(16) \quad \frac{c}{l_w} \leq \frac{1}{390 \left( \frac{\Delta_f R_d R_o}{h_w} \right)} \leq 0.37$$

Figure 3 compares the limits on  $c/l_w$  given by Eqs. (12) to (14) and (16).



**Fig. 3 – Comparison of Simplified Eqs. (12) to (14) for Ductile Coupled Walls (DCW), Ductile Shear Walls (DSW) and Moderately Ductile Shear Walls (MDSW) with Eq. (16) which is equivalent of the ACI 318 limit.**

### **Coupling Beam Rotations**

Building codes generally require diagonal reinforcement in coupling beams with low span-to-depth ratios and high shear stress; but do not place any limits on the inelastic demands put on such coupling beams. Concern has been raised (e.g., Harries [11]) that the rotational demand on coupling beams may greatly exceed rotational capacities. In a recent project [12] to design coupled walls as an undefined system according to the 1997 Uniform Building Code, the chord rotation limits in FEMA 273 were applied, and it was found that the geometry of the coupled walls had to be modified to meet these limits. In the 2004 Canadian concrete code, a new provision was added which requires that the total rotational demand  $\theta_d$  on coupling beams be less than the rotational capacity  $\theta_c$ .

In FEMA 273, the rotational capacity of diagonally reinforced coupling beams is 0.030 and 0.050 radians for Life Safety and Collapse Prevention performance levels, respectively. For conventionally reinforced coupling beams, the limits are 0.015 and 0.030. In the 2004 Canadian concrete code, the coupling beam chord rotations were limited to 0.04 for coupling beams with diagonal reinforcement and 0.02 for coupling beams with conventional reinforcement.

White and Adebar [10] undertook a study to develop a simplified procedure to estimate the maximum demand on coupling beams. They found that coupling beam rotation is proportional to the difference in wall slope and floor slope, where the latter is equal to the relative axial deformation of walls divided by the horizontal distance between the reference points. Using the wall centroids as the reference points gives satisfactory results that are usually safe compared to accounting for the shift in the neutral axis location in walls due to cracking of concrete and yielding of reinforcement.

The wall slope associated with maximum coupling beam rotation, is proportional to maximum global drift, and is much greater than the critical floor slope. Thus, the level of maximum coupling beam rotation occurs near the location of maximum wall slope. This is usually in the lower levels of the coupled walls due to inelastic drift being uniform over the height, and coupling beams pulling back at the top of the walls. Due to the floor slopes, the maximum coupling beam rotations do not necessarily result from the maximum wall slopes during the earthquake.

A simplified procedure that gives reasonable results is to assume that the critical wall slope is equal to the maximum global drift, and the corresponding floor slope is equal to zero. This approach leads to the following equation for estimating the rotational demand on coupling beams which has been incorporated into the 2004 Canadian concrete code:

$$(17) \quad \theta_{id} = \left( \frac{\Delta_f R_0 R_d}{h_w} \right) \frac{\ell_{cg}}{\ell_u}$$

where  $\ell_{cg}$  is the horizontal distance between centroids of the walls on either side of the coupling beams, and  $\ell_u$  is the clear span of the coupling beam between the walls.

### SHEAR STRENGTH OF WALLS

Special design requirements are needed to ensure that a shear failure does not occur in the plastic hinge regions of concrete flexural walls. One such requirement is that the factored shear resistance not be less than the shear corresponding to development of the probable moment capacity accounting for the magnification of shear forces due to inelastic effects of higher modes. Another requirement is that the maximum shear stress is reduced to account for damage from reverse cyclic inelastic rotation of the plastic hinge, which reduces the ability of concrete in that region to resist diagonal compression. Finally, the quantity of transverse reinforcement needs to be increased to avoid accumulative yielding of transverse reinforcement.

It is well known that the reduction in maximum shear stress and increase in transverse reinforcement in the plastic hinge region should be related to the rotational demands on the plastic hinge. Given that designers must already determine the inelastic rotation demands in order to evaluate the ductility (confinement) requirements, this parameter can easily be utilized in shear design.

According to the 2004 Canadian concrete code, the factored shear demand on the plastic hinge region shall not exceed  $0.10 \phi_c f'_c b_w d_v$  unless the inelastic rotational demand on the wall  $\theta_{id}$  is less than 0.015. When  $\theta_{id} \leq 0.005$ , the factored shear demand shall not exceed  $0.15 \phi_c f'_c b_w d_v$ . For inelastic rotational demands between these limits, linear interpolation may be used. The effective shear depth  $d_v$  is equal to the internal flexural lever arm  $jd$  but need not be taken less than  $0.8\ell_w$ .

The shear resistance of a wall shall be taken as:

$$(18) \quad V_r = V_c + V_s$$

where

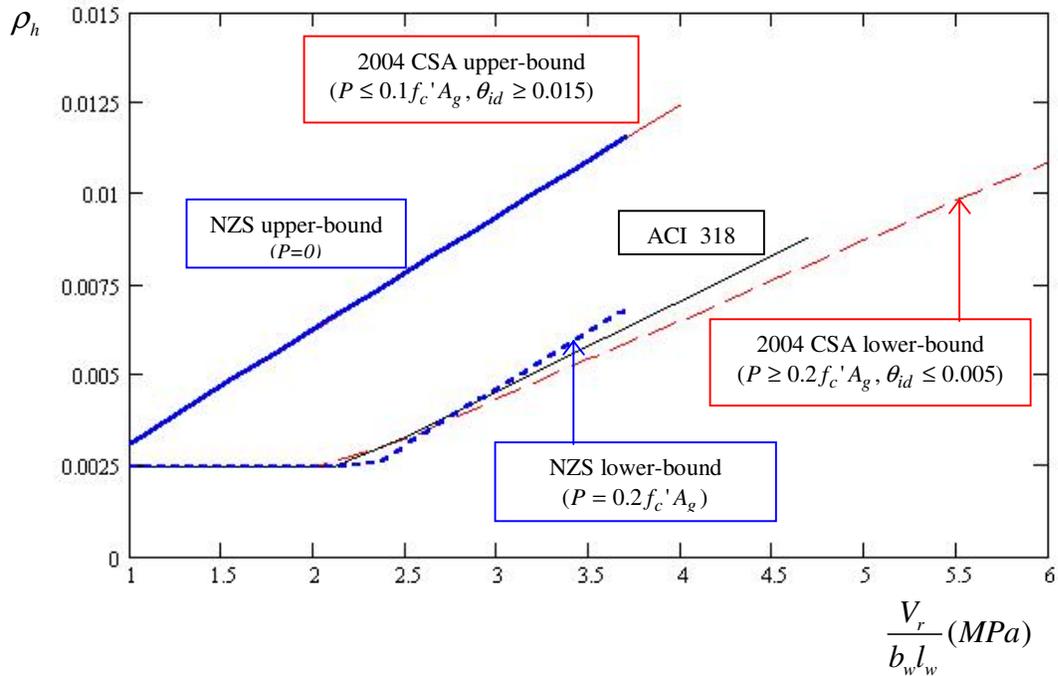
$$(19) \quad V_c = \phi_c \beta \sqrt{f'_c} b_w d_v$$

$$(20) \quad V_s = \frac{\phi_s A_v f_y}{s} d_v \cot \theta$$

The value of  $\beta$  in Eq. (19) shall be taken as zero ( $V_c = 0$ ) unless the inelastic rotational demand on the wall  $\theta_{id}$  is less than 0.015. When  $\theta_{id} \leq 0.005$ , the value of  $\beta$  shall not be taken greater than 0.2. For inelastic rotational demands between these two limits, linear interpolation may be used.

The value of the compression stress angle  $\theta$  in Eq. (20) shall be taken as  $45^\circ$  unless the axial compression force acting on the wall is greater than  $0.1 f'_c A_g$ . When the axial compression is greater than or equal to  $0.2 f'_c A_g$ , the value of  $\theta$  shall not be taken less than  $35^\circ$ . For axial compressions between these limits, linear interpolation may be used.

The designs that result from these new shear design provisions are compared with the results from the current ACI 318 Building Code, and the current New Zealand concrete code [6] in Fig. 4. The plot shows the relationship between shear resistance and quantity of horizontal reinforcement in the wall, expressed as a ratio of steel area to gross concrete area. The end points of the lines indicate the maximum shear stress that is permitted by the respective codes. While a single line describes the ACI 318 provisions, both the 2004 Canadian (CSA) provisions and the New Zealand provisions give a range of results depending on the axial compression force  $P$ , and in the case of the Canadian code provisions, the inelastic rotational demand  $\theta_{id}$ . Thus upper-bound (solid lines) and lower-bound (dashed lines) are shown in the figure and these are similar for the two codes except for the shear stress limits. The maximum shear stress limit in the new Canadian provisions for walls with large inelastic rotational demands is similar to the New Zealand limit, and is less than the ACI 318 limit. On the other hand, the maximum shear stress for walls with small rotational demands is much higher than the New Zealand limit and higher than the ACI 318 limit.



**Fig. 4 – Comparison of horizontal shear reinforcement requirements in the plastic hinge region of flexural walls for  $f'_c = 50$  MPa and  $f_y = 400$  MPa [13].**

## FRAME MEMBERS NOT PART OF SRFS: SLAB-COLUMN CONNECTIONS

The Canadian concrete code requires that all structural members that are subjected to seismically induced lateral deformations, but are not considered part of the Seismic Force Resisting System, must be sufficiently flexible or sufficiently ductile to undergo the displacements. A critical part of the gravity load system of concrete buildings that is particularly sensitive to lateral displacements are flat-plate – column connections.

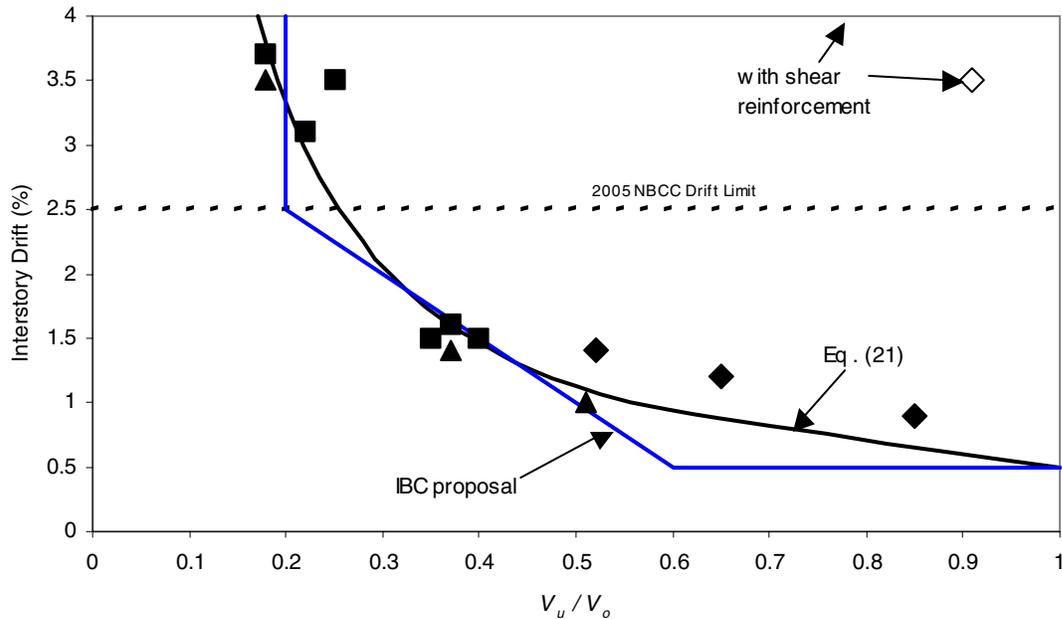
It was first recognized in 1975 [14] that the level of gravity load is the primary variable affecting the lateral displacement capacity of slab–column connections. Based on a review of 23 test results, Pan and Moehle [15] suggested that an interior flat-plate – column connection will perform adequately up to inter-story drift levels of 1.5% if the gravity level shear stress acting on the slab critical section (at  $d/2$  from the column face) does not exceed  $1.5\sqrt{f'_c}$  (psi units). Subsequent experiments on interior slab – column connections [16], [17] and [18] reaffirmed this design recommendation. Megally and Ghali [18] found that exterior slab – column connections can resist approximately 25% higher shear stresses for the same drift level.

The 2004 Canadian code requires that when the maximum two-way shear stress from gravity loads (excluding shear stresses from unbalanced moment) exceeds  $R_E$  times the limiting shear stress for gravity loads, out-of-plane shear reinforcement must be provided in the slab.

$$(21) \quad R_E = \left( \frac{0.005}{\delta_i} \right)^{0.85} \leq 1.0$$

where  $\delta_i$  is the inter-story drift. The 2005 NBCC limits the inter-story drift to 0.025. Figure 5 compares Eq. (21) with the experimental data and the relationship proposed for the International Building Code.

When shear reinforcement is required, it shall be provided such that the maximum gravity load two-way shear stresses (excluding shear stresses from unbalanced moment) does not exceed  $R_E$  times the limiting shear resistance calculated using 50% of the concrete contribution  $v_c$  acting in combination with the “stirrup” contribution  $v_s$  from the out-of-plane shear reinforcement. The factored shear stress resistance of the reinforcement shall not be less than  $0.3\sqrt{f'_c}$ , and the shear reinforcement shall extend a minimum distance of  $4d$  beyond the column face. For post-tensioned slabs, a minimum amount of mild steel bottom reinforcement shall be provided.



**Fig. 5 – Comparison of Eq. (21) with experimental data and the International Building Code proposal.**

### TILT-UP WALLS WITH OPENINGS

Tilt-up concrete walls are commonly used to construct warehouses, shopping centers, office buildings, schools and many other types of buildings. The walls in these buildings often have large openings for windows and doors along an entire side of the building, resulting in wall panels that are actually multi-story frames.

There are two significant differences between tilt-up walls with large openings and typical cast-in-place frames. The first is that the seismic design/detailing procedures used for tilt-up walls is often less stringent than procedures for cast-in-place frames, and as a result, these members have significantly less inelastic drift capacity. The second is that cast-in-place building systems usually have rigid diaphragms, while tilt-up building systems usually have flexible diaphragms. As a result of the flexible diaphragms remaining elastic when the concrete walls yield, tilt-up walls are subjected to much larger inelastic drift demands.

The 2004 Canadian Concrete Code requires that tilt-up wall panels with openings be designed to meet all requirements for moderately ductile cast-in-place frames (ductility force reduction factor  $R_d = 2.5$ ) when the maximum inelastic rotational demand on any part of the wall exceeds 0.02 radians; and in no case shall the inelastic rotational demand on any part of the wall exceed 0.04 radians. As these are the first such provisions for tilt-up frames, the inelastic rotational limits were purposely made less restrictive. It is expected that these limits will be reduced in future editions of the code.

Adebar et al. [19] have developed a simplified method to estimate the inelastic drift demands on concrete tilt-up walls accounting for flexible steel deck diaphragms that remain elastic. These drift demands can be converted into rotational demands on the beams and columns using simple pushover analysis methods.

## REFERENCES

1. Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saaticioglu, M., and Anderson, D.L., "Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada," *Can. J. Civ. Eng.*, Vol. 30, No. 2, April 2003, pp. 308-327.
2. Adebar, P., and Ibrahim, A.M.M.. 2002. Simple nonlinear flexural stiffness model for concrete shear walls, *Earthquake Spectra*, EERI, Vol. 18, No. 3, Aug. 2002, pp. 407-426.
3. Ibrahim, A.M.M., and Adebar, P., "Effective Flexural Stiffness Used for Linear Seismic Analysis of Concrete Walls," *Canadian Journal of Civil Engineering*, In press.
4. CAC. "Explanatory notes on CSA Standard A23.3-94," Part 1, *Concrete Design Handbook*, Cement Association of Canada, Ottawa. 1995.
5. Gonzalez, E., 2001, *Seismic Response of Diagonally Reinforced Slender Coupling Beams*, MASC Thesis, University of British Columbia, Vancouver, British Columbia, Canada, 152 pp.
6. NZS 3101: Part 2. 1995. "Concrete Structures Standard Part 2 – Commentary on the design of concrete structures," *Standards New Zealand*, 264 pp.
7. Mitchell, D., Mutrie, J.G., Adebar, P., and Paultre, P., "Proposed seismic design provisions of A23.3 – Design of Concrete Structures," CSCE conference, Moncton, NB, June 2003.
8. Adebar, P., Mutire, J., and DeVall, R., "Ductility of Concrete Walls: the 2004 Canadian Code," *Canadian Journal of Civil Engineering*, In press.
9. Wallace, J., and Orakcal, K. 2002. "ACI 318-99 provisions for seismic design of structural walls," *ACI Structural Journal*, 99(4), 499-508/
10. White, T. and Adebar, P., "Estimating Rotational Demands in High-rise Concrete Wall Buildings," 13<sup>th</sup> WCEE, Vancouver, BC, Aug. 2004.
11. Harries, K.A., 2001, Ductility and Deformability of Coupling Beams in Reinforced Concrete Coupled Walls, *Earthquake Spectra*, **17**, 457-478.
12. Mutrie, J., Adebar, P., and Leung, K., 2000, Design of High-rise Concrete Shear Wall Buildings: Canadian vs. U.S. Practice, *Proceedings of 2000 Structural Engineers Association of California (SEAOC) Annual Conference*, August, 169-179.
13. Jiao, J.J., "Evaluation Of Shear Requirements For Seismic Walls In Draft 2004 Canadian Concrete Code," M.Eng. Directed Studies Report, Dept. of Civil Eng., Univ. of British Columbia, 2003, 74 pp.
14. Kanoh, Y, and Yoshizaki, S., "Experiments on Slab-Column and Slab-Wall Connections of Flat Plate Structures," *Concrete Journal* (Tokyo), V. 13, June 1975, pp. 7-19.
15. Pan, A., and Moehle, J.P., "Lateral Displacement Ductility of Reinforced Concrete Flat Plates," *ACI Structural Journal*, V. 86, No. 3, May-June 1989. pp. 250-258.
16. Robertson, I.N., and Durrani, A., "Gravity Load Effect on Seismic Behavior of Interior Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 1., Jan.-Feb. 1992, pp. 37-45.
17. Pan, A., and Moehle, J.P., "An Experimental Study of Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 6, Nov.-Dec. 1992, pp. 626-638.
18. Megally, S., and Ghali, A., "Punching Shear Design of Earthquake-Resistant Slab-Column Connections," *ACI Structural Journal*, V. 97, No. 5, Sept.-Oct. 2000, pp. 720-730.
19. Adebar, P., Guan, Z., and Elwood, K., "Displacement-based Design of Concrete Tilt-up Frames Accounting for Flexible Diaphragms," 13 WCEE, Paper 1054, Vancouver, BC, Aug. 2004.