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Design of Gravity-Load Frames in Shear Wall Buildings for Seismic Deformation Demands: The Canadian Code Approach

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ABSTRACT

The Canadian Code requirements for designing cast-in-place reinforced concrete gravity-load resisting frame (GLRF) members for seismic deformation demands have steadily increased in recent editions of the Code and there are new requirements in the upcoming edition. This paper briefly summarizes the Canadian Code requirements for designing the GLRF in concrete buildings and presents new information on how to apply the simplified analysis procedure in CSA A23.3 Clause 21.11. It is recommended that two separate 2D displacement profiles be used to apply the Code defined interstorey drift ratio envelopes, and the procedure for applying these displacement profiles to a 3D model of the building in Etabs is described. The results are presented from case studies on a typical 30-storey highrise concrete shear wall building located in Vancouver. Two different building configurations were considered. In the first, the building has a uniform GLRF over the height, while in the second configuration there is a 1.2 thick transfer slab on level 7. The results of the full-building analysis demonstrates that there are critical "hot spots" in the GLRF. When the GLRF is uniform, the critical spots in the gravity-load columns are at grade level, whereas when there is a significant transfer slab, the critical spots are immediately above and below the transfer slab, as well as at grade level. In order to meet the Canadian Code requirements, design changes had to be made to the gravity-load columns in the case study buildings. The changes include changing the column dimensions to meet the limitations in moderately ductile columns, increasing the flexural resistance of the columns by increasing the amount of vertical reinforcement and increasing the shear strength of the columns by adding transverse shear reinforcement.

Keywords: Building codes, Concrete buildings, Gravity-load resisting frame, Structural engineering practice.

INTRODUCTION

Like most building codes, the Canadian Code requires that a clearly defined seismic force resisting system (SRFS) be designed to resist 100% of the earthquake loads, and that the stiffness imparted to the structure from structural elements that are part of the gravity-load resisting frame (GLRF) not be used to reduce earthquake deflections. When the GLRF is a flexible structure, the two parts of the building can be designed separately – SFRS is design to resist earthquake (and wind) demands, while the GLRF is designed only for gravity loads.

In the seismically active regions of Canada, most highrise buildings are concrete shear wall buildings, and where the seismic demands are large, such as on the west coast of Canada, modern highrise buildings are core wall buildings (with shear walls arranged in a central core). When the gravity-load resisting frame (GLRF) is constructed of cast-in-place reinforced concrete, special attention needs to be paid to ensuring that the all members in the GLRF can tolerate the lateral displacements of the building due to the design level earthquake ground motions because typical reinforced concrete GLRF do not have the flexibility to tolerate the type displacement demands that occur in highrise concrete buildings. Damage of the reinforced concrete GLRF due to deformation demands from lateral movement of the building during an earthquake can cause complete collapse of the building or at the very least, can make the building uninhabitable after an earthquake. Concern about the deformation capacity of gravity-load resisting frame members has increased more recently as these frames have become more complex and more irregular in modern highrise buildings.

The Canadian Code requirements for designing cast-in-place reinforced concrete gravity-load resisting frame members for seismic deformation demands have steadily increased in recent editions of the Code and there are new requirements in the upcoming edition. This paper briefly summarizes the Canadian Code requirements for designing the GLRF in concrete buildings and presents new recommendations on how to apply these requirements to a typical core wall building. Results are presented from some case studies that demonstrate how the Canadian Code requirement can influence the design of the concrete gravity-load resisting frame members in typical highrise buildings on the west coast of Canada.

CANADIAN CODE REQUIREMENTS FOR GRAVITY-LOAD RESISTING FRAMES

In this paper, the 'Canadian Code' refers to two documents. The first is the national model code, National Building Code of Canada (NBC), which is adopted provincially, e.g., the British Columbia Building Code (BCBC). The second document is the Canadian Standard CSA A23.3-14, Design of Concrete Structures (referred to here as simply CSA A23.3).

NBC requirements

The NBC requirements pertaining to the design of gravity-load resisting frames (GLRF) can be summarized as: (1) The SFRS must be designed to resist 100% of the earthquake loads. (2) Elements of the GLRF must be investigated and shown to behave elastically or to have sufficient non-linear capacity to support their gravity loads while undergoing earthquake-induced deformations. (3) Stiffness imparted to the structure from GLRF must not be used to resist earthquake deflections but must be accounted for (i) in determining the earthquake loads if the added stiffness decreases the fundamental lateral period by more than 15%, (ii) in determining the irregularity of the structure, except the additional stiffness must not be used to make an irregular SFRS regular or to reduce the effects of torsion, and (iii) in designing the SFRS if inclusion of the elements in the GLRF have an adverse effect on the SFRS.

The requirement that the SFRS be designed to resist 100% of the earthquake loads and the requirement that the structural modeling include the stiffness of all elements that influence the lateral stiffness of the building means that two different structural models are required to design a building – the first model of the building includes only the stiffness of the SFRS, while the second model also includes the stiffness of the GLRF.

The flexibility of soil or rock under a shear wall foundation will reduce the seismic demands on the SFRS (shear walls); but will increase the seismic demands on the GLRF. Thus, in 2015 a new provision was added to NBC that requires the increased displacements of the structure resulting from foundation movement be accounted for. The bearing stress distribution in soil or rock that is used to determine the factored overturning resistance of the foundation influences how much the foundation will need to rotate in order to develop the calculated resistance. If the factored bearing stress at one end of the footing is assumed to be over a short length, then the uplift and rotation of the footing can be very significant.[1] NBC refers to the procedures given in CSA A23.3 for determining how much a footing rotates, which are based on Ref. [1].

In the 2020 edition, an additional performance requirement was added to NBC for SFRS and GLRF in post-disaster buildings and high-importance category buildings, and for the GLRF in tall buildings (height above grade more than 30 m) in seismically active regions (SC3 and SC4). For tall concrete buildings on the west coast, all elements of the GLRF must be investigated and shown to behave elastically when the building is subjected to ground motions having a 10% probability of exceedance in 50 years. New requirements in the 2024 edition of CSA A23.3 will provide guidance on how that is to be achieved.

CSA A23.3 requirements

CSA A23.3 defines the procedures to be used to confirm that the elements of the GLRF in concrete buildings either behave elastically or to have sufficient non-linear capacity to support their gravity loads while undergoing earthquake-induced deformations. The most general approach to determining the demands on the GLRF of a concrete shear wall building is response history analysis using a nonlinear model of the complete building, or a nonlinear model of the SFRS and a linear model of the GLRF. As nonlinear analysis is rarely used for design in Canada, CSA A23.3 provides simplified procedures to determine the demands on the GLRF due to the nonlinear response of the SFRS.

Shear walls apply bending demands to the gravity-load resisting columns and walls through the connecting floors in two different ways. First, due to the very high in-plane rigidity of the closely spaced floor slabs, the gravity-load columns and walls are forced to have the same deflected shape, and hence the same curvature, as the shear walls. Throughout most of the height of the building, the curvature demands in the shear walls are low compared to the curvature capacity of the gravity-load columns and walls. This is not the case, however, in the plastic hinge region of the shear walls, and important requirement in CSA A23.3 is to confirm that the curvature capacity of all gravity-load columns and walls is greater than the curvature demand on the concrete shear walls in the plastic hinge region.

The second way that shear walls impose demands on the GLRF is through frame action resulting from building drifts. Depending on the relative bending stiffness of the horizontal frame members (slabs and beams) connecting vertical frame members (walls and columns), the building interstorey drift ratios generate bending moments and shear forces in the gravity-load columns and walls.

CSA A23.3 Clause 21.11

Clause 21.11 of CSA A23.3 prescribes the procedures to be used to determine the demand on GLRF members due to frame action between the shear walls and the GLRF members. The code specifies requirements for two approaches. The first are general analysis requirements, while the second are the requirements for a simplified procedure. The background to these procedures are given in Ref. [2].

The general analysis procedure requires that the complete structure be displaced laterally to the design displacements determined from an analysis in accordance with the NBCC, incorporating the effects of torsion, including accidental torsion, and accounting for foundation movements. CSA A23.3 specifies that the inelastic displacement profile of the SFRS must be accounted for; but provides no guidance on how that is to be done. The difficulty of accounting for the inelastic displacement profile of the SFRS discourages designers from utilizing the general analysis procedure.

In the simplified procedure, the shear force and bending moments induced in members of the GLRF are determined by subjecting the entire building, SFRS and GLRF, to the interstorey drift ratio for that level given in Figure 1. The deflection Δ_{max} used to calculate the global drift ratio Δ_{max}/h_w given in Fig. 1 is the design lateral deflection at the top of a specific GLRF determined from an analysis in accordance with the NBCC, incorporating the effects of torsion, including accidental torsion, and accounting for foundation movements.

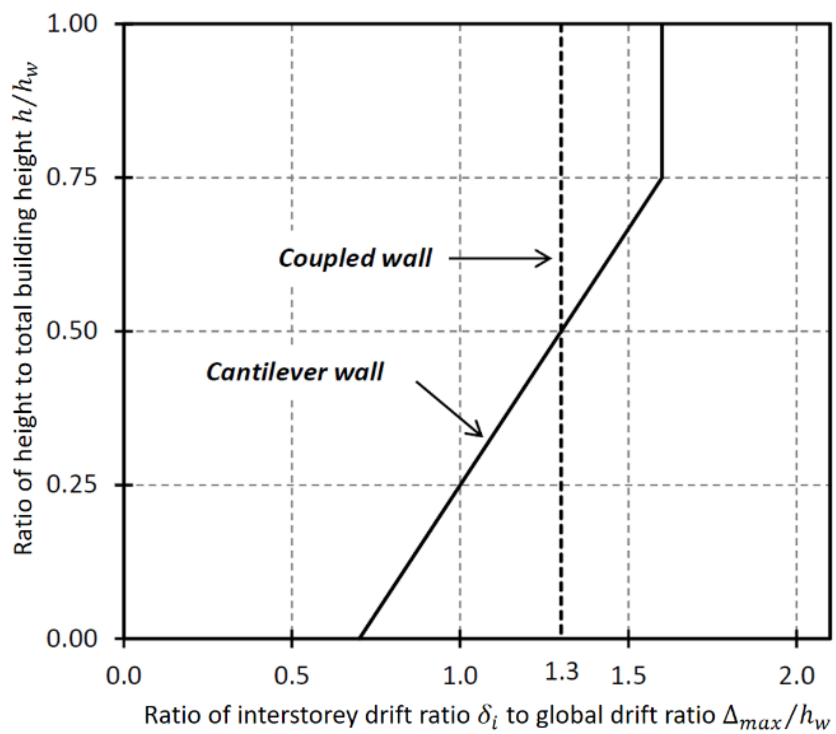


Figure 1. Envelopes of minimum interstorey drift ratios over the building height prescribed for concrete shear wall buildings in the simplified analysis procedure of CSA A23.3 Clause 21.11.

SIMPLIFIED ANALYSIS OF SHEAR WALL BUILDINGS

Displacement profiles

The interstorey drift ratios given in the envelopes in Fig. 1 are from different modes of the shear wall response [3]. The interstorey drifts at the top of the wall are primarily from first mode, while the interstorey drifts at the lower floors are from the second mode and other higher modes, as well as due to shear strain in the plastic hinge region of the shear walls. [4] Applying a displacement profile with the maximum interstorey drift at every level will result in an overly conservative estimate of the axial load demands in the gravity-load columns. Thus it is recommended to apply to displacement profiles separately, one defining the displacement profiles over the top half of the building, and the second defining the displacement profile over the bottom half of the building. The recommended profiles are summarized below. Also given below are the displacement profiles giving the maximum interstorey drift at all levels of the building.

Cantilever Wall – displacement defined over top half of wall

h/h_w	Δ/Δ_{max}
0.50	0.2375
$\geq 0.50; \leq 0.75$	$0.2375 + 1.30\,(h/h_w - 0.5) + 0.60(h/h_w - 0.5)^2$
$\geq 0.75; \leq 1.0$	$0.60 + 1.60\,(h/h_w - 0.75)$

Cantilever Wall – displacement defined over bottom half of wall

h/h_w	Δ/Δ_{max}
$\geq 0; \leq 0.50$	$0.70\,(h/h_w) + 0.60(h/h_w)^2$

Cantilever Wall – displacement defined over full height of wall

h/h_w	Δ/Δ_{max}
$\geq 0; \leq 0.75$	$0.70\,(h/h_w) + 0.60(h/h_w)^2$
$\geq 0.75; \leq 1.0$	$0.8625 + 1.60\,(h/h_w - 0.75)$

Coupled Wall – displacement defined over top half of wall

h/h_w	Δ/Δ_{max}
0.50	0.35
$\geq 0.50; \leq 1.0$	$0.35 + 1.30\,(h/h_w - 0.50)$

Coupled Wall – displacement defined over bottom half of wall

h/h_w	Δ/Δ_{max}
$\geq 0; \leq 0.50$	$1.30\,(h/h_w)$

Coupled Wall – displacement defined over full height of wall

h/h_w	Δ/Δ_{max}
$\geq 0; \leq 0.10$	$1.30\,(h/h_w)$

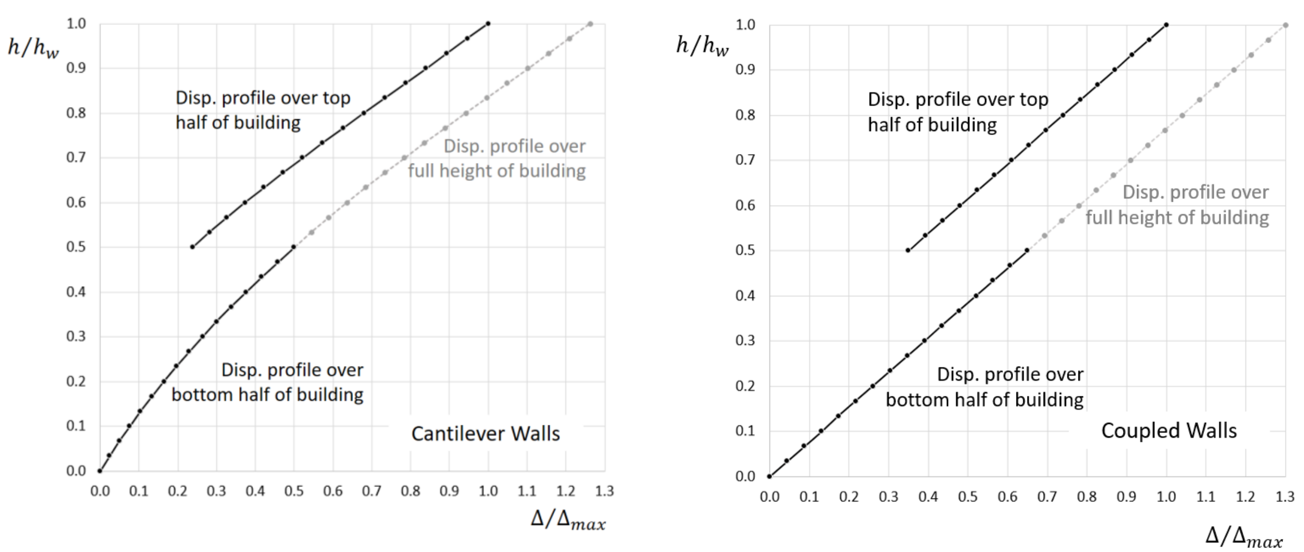


Figure 2. Recommended two-part displacement profiles giving interstorey drift profiles defined in CSA A23.3 Clause 21.11.

Application of displacement profiles using Etabs

The two-dimensional displacement profiles presented above are applied to the complete model of the building with the stiffness of all SFRS and GLRF members included. The displacements are imposed on the building model using fictitious members that have infinite axial rigidity and bending moment releases at both ends (the members resist only axial load). The members are oriented parallel to the direction of applied seismic demands, i.e., either parallel with the coupled walls or the cantilever walls. The axially-rigid members are used to apply horizontal force to the building model at the point of contact with the rigid diaphragms (floor levels). Pin supports are defined at the other end of the axially-rigid members, and the displacement profiles are imposed in Etabs using the '*Ground Displacement'* command for the pin supports.

As described above, there are three different types of displacement profiles: (i) displacement profile over full height; (ii) displacement profile over top half of building, and; (iii) displacement profile over bottom half of building. When the displacement profile is defined over just one half of the building height, the remainder of the building levels are allowed to deform freely.

Two 2D displacement profiles are applied on two parallel gravity-load frames in order to define the complete 3D displacement demands on the 3D building model. One approach is to apply a uniform profile of horizontal displacements across the entire building. A more efficient approach is to investigate two different gravity-load frames at the same time by applying two different (2D) vertical displacement profiles along two gravity load frames. Due to inherent and accidental torsion, gravity-load resisting frames in a building typically have different maximum displacement demands. The gravity-load frames along the outside of the building typically have the largest displacement demands.

CASE STUDY

Description of building

A case study was conducted using a typical 30-storey highrise concrete shear wall building located in Vancouver. Two different cases were considered. In the first, the building had a uniform gravity-load resisting frame, while in the second case, a 1.2 thick transfer slab was added at level 7; otherwise the building examples were identical.

The buildings had 30 levels above grade, and 5 levels below grade. The typical clear storey height above grade is 2.59 m, while at grade, the clear storey height is 4.21 m. All floors above grade are 190 mm thick concrete flat plate slabs and the floor slab at grade is 250 mm thick. The overall height of the building from the top of the floor slab at Level 1 (grade) to the top of the roof slab is 85.080 m

Fig. 3 shows a plan view of a typical floor level above grade. The overall dimensions of the suspended floor plates are 25.90 m x 25.90 m. The central core has three wall piers. In the cantilever wall (North-South) direction the overall dimensions of the core is 8.94 m, while in the coupled-wall (East-West) direction, the overall dimensions of the core is 7.72 m. The Etabs model of the building without the stiffness of the GLRF members included, indicates the fundamental lateral periods are 6.44 s in the coupled-wall direction, and 4.82 s in the cantilever wall direction.

There are 16 gravity-load columns that support the floor slabs as shown in Fig. 3. For simplicity in this case study, all gravity-load columns are identical at a particular level in the building. Figure 4 summarizes the column designs. The column dimensions are 350 x 1000 mm from the foundation to Level 11, and 300 x 750 mm in the upper levels.

Overview of analysis results

The results from the analysis of the case study building with a uniform GLRF are summarized in Figs. 5 and 6, while the results from the analysis of the building with a transfer slab are summarized in Figs. 7 and 8.

Figure 5 presents the variation of shear force and bending moment over the height of the building from the three different displacement profiles. The light blue lines are the variation of shear force or bending moment resulting from the imposed displacement over the top half of the building, while the dark blue lines are the variation of shear force or bending moment resulting from the imposed displacement over the bottom half of the building. The red dashed lines are the variation of shear force or bending moment resulting from the displacement profile that causes the interstorey drift envelope over the full height of the building.

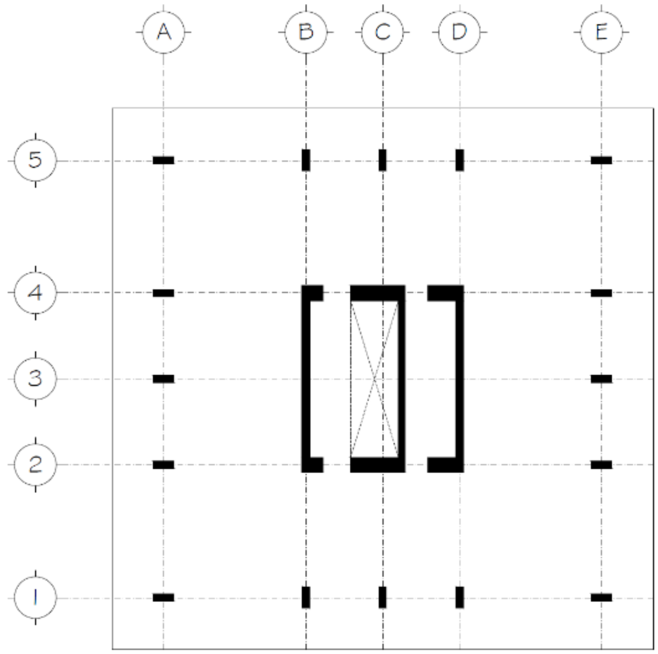
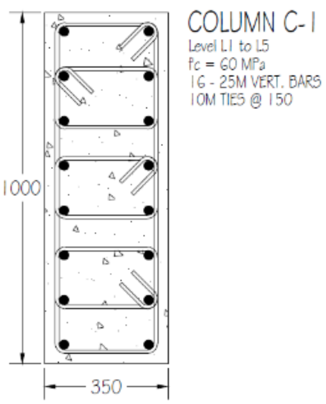


Figure 3. Plan view of 30-storey case study building showing outline of floor plate, central core and arrangement of gravity-load columns; Note there five different GLRF in each direction.



Levels	f'c [MPa]	Dim. [mm]	Vert. reinf.	Ties
L21 - L30	35	300 X 750	8 - 25M	10M @ 300
L15 - L20	50	300 X 750	8 - 25M	10M @ 300
L12 - L14	50	300 X 750	12 - 25M	10M @ 300
L6 - L11	60	350 X 1,000	16 - 25M	10M @ 250
L1 - L5	60	350 X 1,000	16 - 25M	10M @ 150
P5 - P1	80	350 X 1,000	16 - 25M	10M @ 250

Figure 4. Summary of gravity-load column design.

Comparing the red dashed lines with the blue lines, it can be seen that the red lines give the same result as the light blue lines in the upper half of the building and the same as the dark blue lines in the lower half of the building. Owing to the discontinuity in defined displacements at mid-height, the blue lines have increased bending moments at mid-height, whereas the red lines do not show this. This suggests that the single displacement profile over the full height can be used to make a good estimate of the shear force and bending moment demands in the gravity-load columns. Note that there are increased shear forces and bending moments at the levels where the column dimensions change due to the sudden change in column bending stiffness.

Figure 6 presents the variation of axial load in the gravity-load columns due to vertical shear forces in the slabs from lateral displacement of the building. This effect is sometimes referred to as 'unintentional outrigger effect.' In this case, the red dashed line gives the same result as the light blue line in the top half of the building, but gives overly conservative results in the bottom half of the building. Figure 6 illustrates why the two separate half-profiles must be used. The correct estimate of axial load is the envelope of the two half displacement profiles, i.e., the larger value from the two blue lines. The displacement profile over the top half gives the larger axial load value above about 18 m elevation, while the displacement profile over the bottom half of the building is the larger value in the lower elevations.

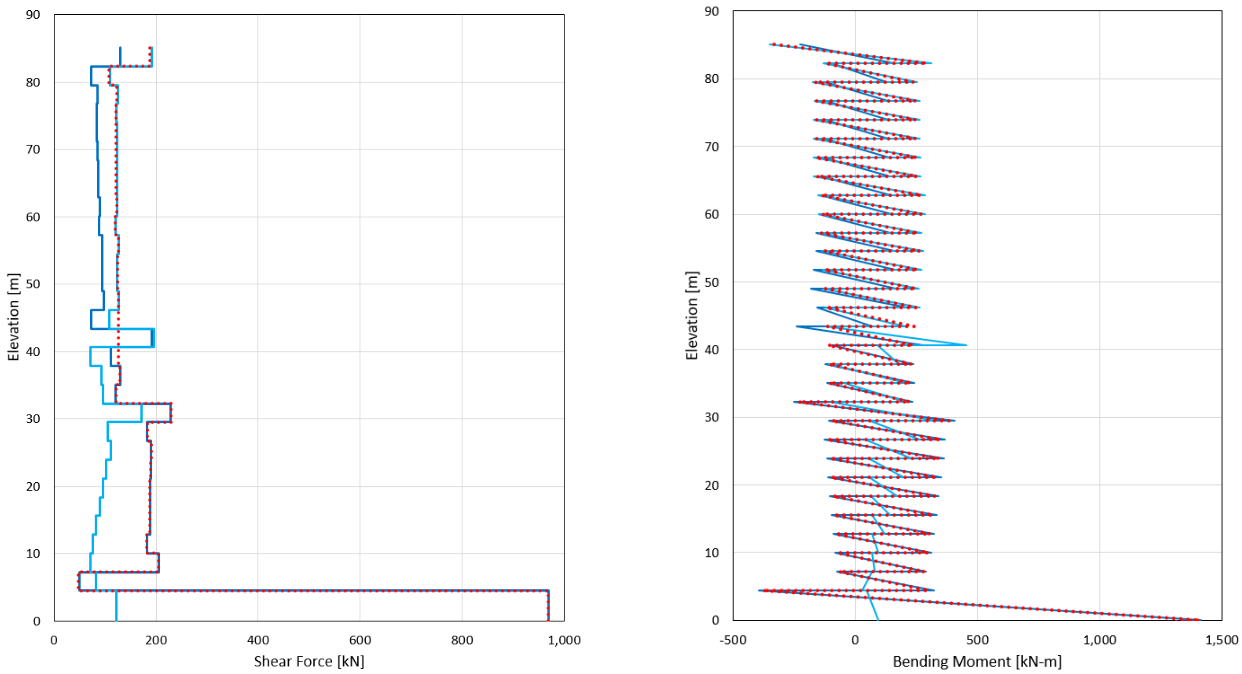


Figure 5. Results from uniform GLRF: variation of shear force (left) and bending moment (right) in gravity-load column; light blue – disp. over top half; dark blue – disp. over bot. half; red – disp. over full height.

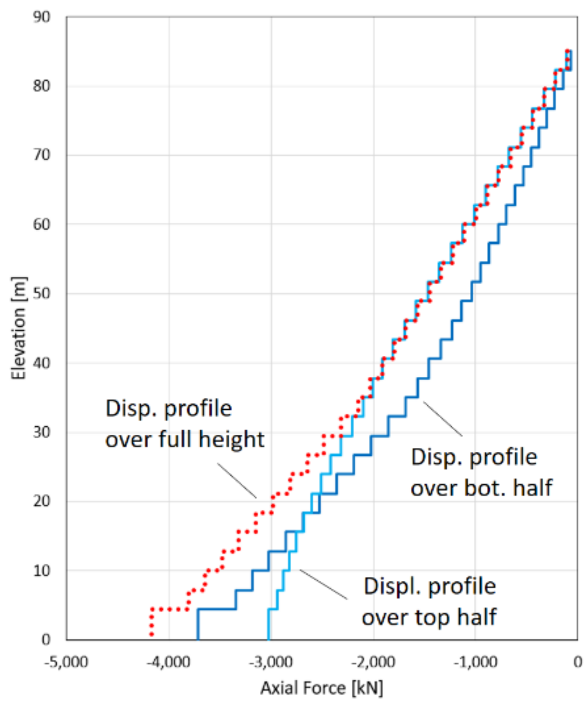


Figure 6. Variation of axial load in column in building with uniform GLRF from three different lateral displacement profiles.

Figure 7 presents the variation of shear force and bending moment over the height of the building with the transfer slab from the three different displacement profiles. In this case very large shear forces and bending moments are induced in the gravity-load columns immediately above and below the transfer slab. These forces are significantly larger than the forces that occur at grade level. Figure 8 presents the axial loads that develop in the gravity-load columns on the compression side of the building. Due to the very large out-of-plane bending stiffness of the 1.2 m deep transfer slab, very large vertical shear forces develop in the transfer slab and this causes very large axial loads in the gravity-load columns at the level of the transfer slab.

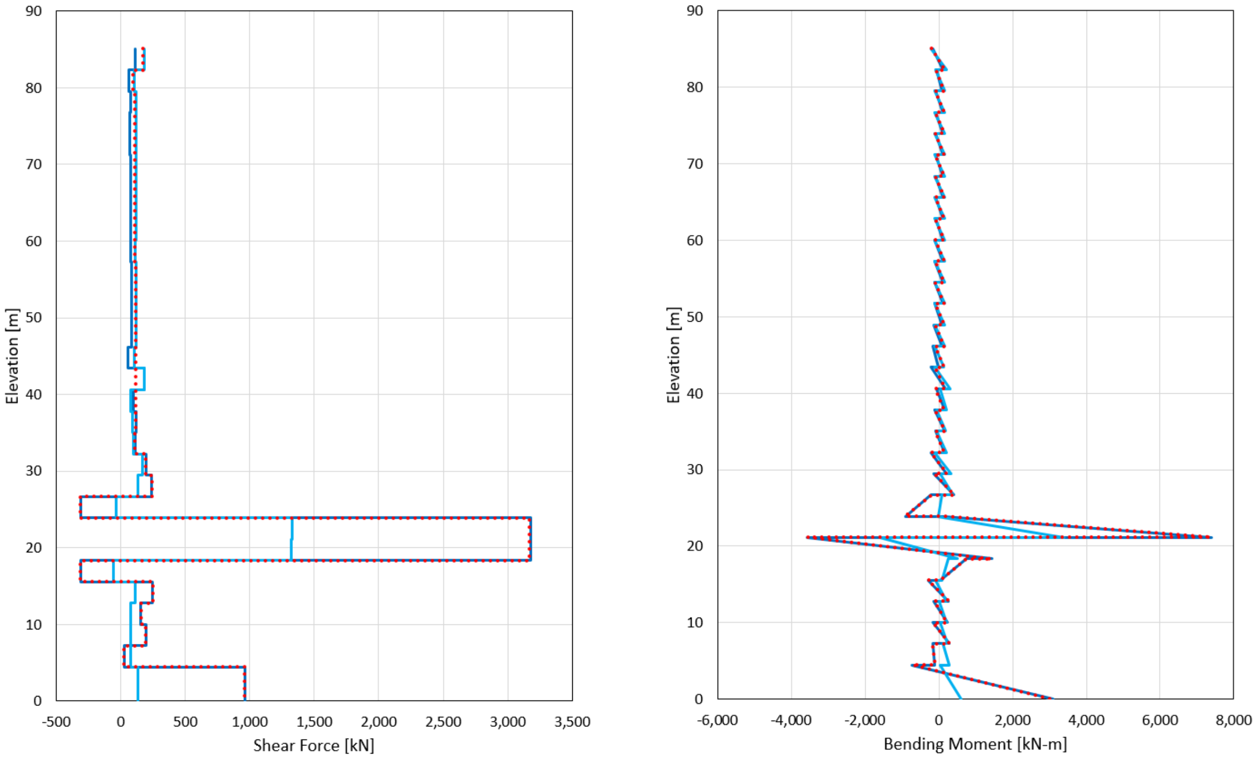


Figure 7. Results from building with transfer slab at Level 20: variation of shear force (left) and bending moment (right) in gravity-load column; light blue – disp. over top half; dark blue – disp. over bot. half; red – disp. over full height.

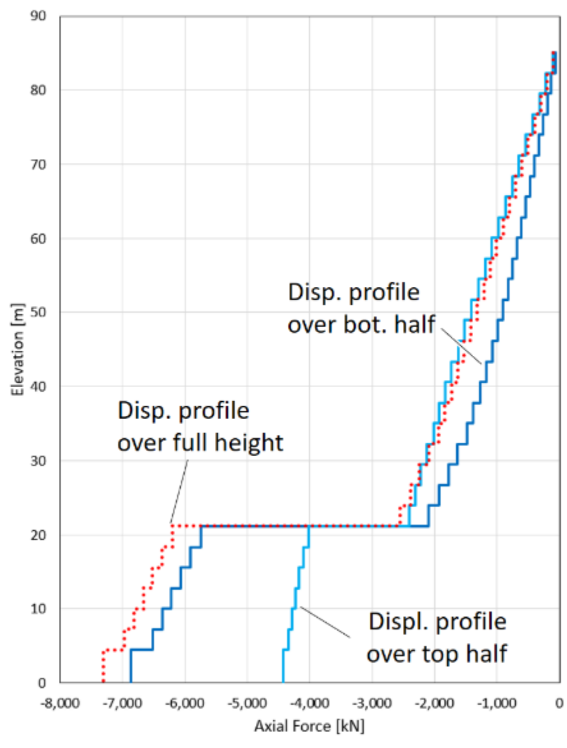


Figure 8. Variation of axial load in gravity-load column in building with transfer slab at Level 20.

Required design changes in gravity-load columns

In the CSA A23.3 simplified (force-based) procedure, the calculated induced bending moment determined from a linear analysis is limited depending on the type of column and level of applied axial compression. For a column that meets all the design requirements for a ductile moment-resisting frame member and has a low level of axial compression (less than 0.2 $f_c' A_g$), the induced bending moment is permitted to be as large as 5.0 times the factored bending resistance M_r. The factored resistances are used to account for the uncertainty in displacement demands – resistances are reduced rather than displacement demands increased, and multiples of factored resistance are used as indicators of inelastic displacement demands. On the other end of the spectrum, for a simple tied gravity-load "wal-lumn" (column with large ratio of larger cross-sectional dimension to smaller cross-sectional dimension like a wall), such as the columns in the case study building, with high axial load (greater than $0.4 f_c' A_g$), the induced bending moment is limited to only $1.0M_r$. There are six additional cases with different column types, axial load levels and correspondingly different maximum permitted induced bending moments between the two bounds described above.

The results of the analysis presented above indicates the gravity-load columns require significant design changes in order to meet the CSA A23.3 requirements. For the building with the uniform GLRF, the design changes all occur on Level 1 where the bending moment demands are largest as shown in Fig. 5. The 12 columns that are part of a GLRF that includes the core (Columns B1, C1, D1, B5, C5, D5, A2, A3, A4, E2, E3, E4 in Fig. 3) require design changes. The four corner columns A1, A5, E1, E5 do not require design changes because of the lower induced demands resulting from the higher flexibility of the floors in the GLRF along the outside edge of the building (Frames 1, 5, A and E in Fig. 3).

The bending moments induced in the gravity columns are significantly more critical in the coupled wall direction (Frames 2, 3 and 4) for two reasons. Firstly, the core of a typical Vancouver highrise shear wall building, such as that shown in Fig. 3, is more flexible in the coupled-wall direction compared to the cantilever direction. For the current building, the fundamental lateral period is 6.44 s in the coupled-wall direction, and 4.82 s in the cantilever direction. Thus, the design displacement, which is the input to the calculation of the interstorey drift envelope, is larger. For the current, the global drift demand is 0.0093 in coupled-wall direction and 0.0076 in cantilever wall direction. The second reason the bending moments induced in the gravity columns are more critical in the coupled wall direction is that the ratio of interstorey drift ratio to global drift ratio at the first level above the base is much larger in the coupled wall direction than in the cantilever wall direction (1.3 in the coupled-wall direction versus 0.7 in the cantilever wall direction, see Fig. 1).

The columns on the compression side of the building are more critical in the linear static analysis of the building because their effective flexural rigidity is higher, and therefore attract larger bending moments due to the lateral displacement of the building. Also, the additional axial load from lateral displacement of the building means the factored bending resistance is lower and the CSA A23.3 limit on the ratio of bending moment demand to factored resistance is lower. Of course, the tension side becomes the compression side when the lateral movement of a building changes direction in an earthquake.

In the cantilever wall direction of the building, the dimensions of the columns had to be modified to meet the dimensional limitations for moderately ductile columns, which is that the ratio of smallest cross-sectional dimension to perpendicular dimension not be less than 0.4. The column dimensions were changed to 400 mm x 1000 mm. In the coupled-wall direction, the axial resistance of the columns also had to be increased in order for the columns to tolerate the induced bending moment. All of the columns had sufficient shear capacity.

For the building with the transfer slab, the results are very sensitive to the level of cracking in the transfer slab. If a low level of cracking is assumed, the forces induced in the columns immediately above and below the transfer slab require significant redesign. The most severe demands occur on the columns immediately above the transfer slab on the compression side of the building. Very large shear force demands occur due to large bending moment gradient. Shear force demands greater than 10 times the factored shear resistance were observed in some analyses.

The reason for the very large demands on the level immediately above the transfer girder on the compression side of the building can be explained by the deflected shape of the GLRF. When the thick floor slab is very stiff, the gravity-load column does not have sufficient stiffness to influence the slope of the floor slab. As a result, the column is essentially a pin support to the thick slab, and the slope of slab immediately above the column on the compression side of the building is in opposite direction of the slope of the rest of the building. For example, if lateral drift causes a clockwise rotation of the building, the opposite end of the thick slab rotates in the counter-clockwise direction. With the column supporting the slab also rotating clockwise, the counter-clockwise rotation of the floor slab causes severe bending moments and bending moment gradients (shear forces) in the column immediately above the transfer slab.

The thick transfer slab also causes large axial forces in the gravity-load columns due to lateral earthquake displacements when the transfer slab has a low degree of cracking. The induced earthquake forces (1.0E) due to lateral displacement of the building must be considered in combination with the gravity loads (either 1.0D or 1.0D + 0.5L + 0.25S). In some cases the combined axial compression is larger than the factored axial resistance of the columns. In the reverse direction, the uplift forces in the column on the tension side of the building are sufficient to counteract the gravity loads, and in some cases the net tension force exceeded the factored tension capacity of the columns suggesting all the vertical reinforcement would yield in tension.

It is interesting to note that the axial demands on the columns due to lateral movement of the building are more severe in the cantilever wall direction of the building. This is because the core is larger in that direction and thus the span of the slab from the core to the gravity-load columns is shorter. The vertical forces induced in the columns, which is equal to the vertical shear forces in the slab, are very sensitive to the span of the floor slab.

CONCLUSIONS

CSA A23.3 defines the procedure to be used to confirm that the elements of the gravity-load resisting frame (GLRF) in concrete buildings either behave elastically or to have sufficient non-linear capacity to support their gravity loads while the building undergoes the earthquake-induced movements. The simplified procedure in Clause 21.11 of CSA A23.3 prescribes interstorey drift envelopes that are to be used to determine the seismic demands on the GLRF members; however, designers of concrete buildings have struggled with how to analyze the GLRF subjected to these interstorey drift values. This paper presents a recommended procedure for how that is best done. Two separate 2D displacement profiles, one over the top half of the building and the other over the bottom half, are applied to each GLRF in the building. Axially-rigid members with moment releases at both ends are used to apply horizontal forces to a 3D building model of the building at the floor levels. The displacement profiles are imposed in Etabs using the '*Ground Displacement'* command for the nodes at the pin supports of the axially-rigid members. A single displacement profile with the maximum interstorey drift at every level applied at the same time can be used to make a simple estimate of the bending moments and shear forces induce into the gravity-load resisting columns; however, the two separate displacement profiles must be used in order to not over predict the level of axial load that is induce into the columns from vertical shear in the floor slabs.

Case studies were conducted on a typical 30-storey Vancouver highrise building. In one case the building had a uniform GLRF, while in a second case, the building had a thick transfer slab at Level 7. When the building had a uniform GLRF, the critical condition is bending moment demands on the gravity-load columns at Level 1. The column designs had to be modified at Level 1 in order for the columns to have sufficient inelastic bending capacity to tolerate the horizontal displacements of the building. The coupled-wall direction of the building was found to be more critical because the building is more flexible in that direction and the ratio of interstorey drift to global at the base of the building being larger in that direction.

When a building has a thick slab such as a transfer slab, the results are very sensitive to the level of cracking in the slab. The most severe demands in the GLRF occur in the columns immediately above the transfer slab. Very large shear forces and bending moments are induced at this level requiring significant changes to the design of the columns at this level. Large demands also occur in the first level below the transfer slab and at Level 1.

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