DESIGN OF GRAVITY-LOAD RESISTING FRAMES FOR SEISMIC DISPLACEMENT DEMANDS

P. Adebar,¹ R. DeVall,² and J.G. Mutrie,³

ABSTRACT

New requirements have been developed for the design of members not designated as part of the seismic-force-resisting system in the 2014 edition of CSA A23.3, which is the Canadian equivalent to ACI 318. For shear wall buildings, which is the predominate type of building constructed in Canada, an interstory drift envelope is defined as a function of the global drift demand determined at the top of the gravity-load frame from an analysis that includes the effect of torsion. The variation of interstory drift over the upper stories was developed from the results of numerous nonlinear dynamic analyses of shear wall buildings. The prescribed envelope also accounts for the nonlinear shear deformation in the plastic hinge region of shear walls as was observed in experiments. This nonlinear shear deformation, which currently is not accounted for in state-of-the-art nonlinear analysis of shear walls, greatly increases the interstory drift demands on the gravity-load resisting columns over the plastic hinge region of shear walls. The seismic design requirements for gravity-load resisting columns and bearing walls depend on the inelastic flexural deformation demands on the member. When seismic demands on gravity frames are determined using a linear model, the requirements are determined from how much the induced bending moment exceeds the factored bending resistance of the member. The limit ranges from 5.0 to 0.5 times the factored bending moment resistance for columns from special moment-resisting frames to thin walls with a single layer of reinforcement, respectively. The detailing requirements for gravity-load resisting beams are also based on how much the calculated bending moment exceeds the factored bending resistance. Factored resistances are used as a simple way to compensate for uncertainty in displacement demands. New axial load restrictions have been placed on columns and bearing walls with a minimum dimension less than 300 mm to account for the fact that these members can suddenly lose all vertical load carrying capacity.

¹ Professor, Dept. of Civil Engineering, The University of British Columbia, Vancouver, BC, Canada, V6T 1Z4.
² Senior Structural Consultant, Read Jones Christoffersen Ltd., Vancouver, BC, Canada, V6H 3X8.
³ Senior Structural Consultant, Jones Kwong Kishi, North Vancouver, BC, Canada, V7P 3P7.

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Introduction

Observations from past earthquakes have shown that the collapse of buildings is often triggered by failure of structural members that are not part of the seismic-force-resisting system (SFRS), such as gravity-load columns. As a result, model building codes such as the International Building Code (IBC) in the US or the National Building Code of Canada (NBCC) [1] require that all structural members not designated as a part of the SFRS be designed to support the gravity loads while subjected to the design seismic displacements. In regions of high seismic hazard in the US, it is common practice for the gravity-load columns to be similar to the columns in special moment-resisting frames – square cross-sections with a large amount of column ties. In regions of high seismic risk in Canada, on the other hand, it is not uncommon for gravity-load

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1 Professor, Dept. of Civil Engineering, The University of British Columbia, Vancouver, BC, Canada, V6T 1Z4.
2 Senior Structural Consultant, Read Jones Christoffersen Ltd., Vancouver, BC, Canada, V6H 3X8.
3 Senior Structural Consultant, Jones Kwong Kishi, North Vancouver, BC, Canada, V7P 3P7.

columns to have very few column ties and for the cross sectional dimensions to be more like a wall than a column, e.g., 250 mm x 1500 mm. Such columns have very little flexibility about the strong axis of bending. In some buildings, thin bearing walls with a single layer of reinforcement are used as the vertical load carrying members. Gravity-load resisting columns and bearing walls in Canada often do not extend over the full height of the building. They “transfer” at one or sometimes multiple levels by transfer girders that may be connected to the SFRS. As a result of the concern about the seismic resilience of gravity-load resisting frames in Canada, many additional requirements have been developed for the design of these members for Clause 21.11 in the 2014 edition of CSA A23.3 [2], which is the Canadian equivalent to ACI 318. This paper provides a summary of these new requirements and briefly provides the background research that led to the development.

The requirements for structural members that are not part of the seismic-force-resisting system in Clause 21.11 of CSA A23.3-2014 will need to be applied independent of the ductility-based forced reduction factor used to design the SFRS, i.e., regardless of whether the designer chooses to design a highly ductile (special) SFRS or a conventional SFRS with higher strength and less ductility. The provisions will not need to be applied to buildings located where the seismic hazard is very low – defined as when the design spectral acceleration at 0.2 seconds times the importance factor is less than 0.35g. The provisions also will not need to be applied when the maximum interstorey drift ratio is less than 0.5% at every point in a building. The interstorey drift ratios are to be determined from a linear dynamic analysis accounting for torsion, including accidental torsion, and accounting for foundation movements.

Analysis of Gravity-Load Resisting Frames

An analysis must be done to determine the forces and deformations induced in the gravity-load resisting frame members due to the seismic deformation demands on the SFRS. In concept, this involves displacing the complete structure – SFRS and gravity-load resisting frame – to the design displacement. The yielding that occurs in the SFRS before it reaches the design displacement causes a concentration of deformation demands at plastic hinge locations. The influence of the resulting inelastic displacement profile of the SFRS must be accounted for when determining demands on the gravity-load frame. The best way to accomplish this is to do a nonlinear analysis of the complete structure; however a linear model with appropriately reduced section properties at plastic hinge locations can be used to estimate the inelastic displacement profile for shear wall buildings [3]. An important issue is what stiffness should be used for the gravity-load resisting frame members. Lower-bound estimates of effective stiffness are commonly used for the SFRS to make a safe estimate of the design displacement; but higher estimates of effective stiffness of the gravity-load resisting frame members must be used to make a safe estimate of the forces induced in these members by the inelastic displacement profile of the SFRS. The increased displacements of the SFRS due to foundation movements must be accounted for. A companion paper [4] explains how this can be done in a simple way.

Simplified Nonlinear Analysis of Shear Wall Buildings

Most Canadian designers do not use any nonlinear analysis for the design of concrete buildings. Thus a simplified analysis procedure was developed for gravity-load resisting frames in shear
wall buildings. The simplified analysis procedure was developed from the results of numerous nonlinear analyses of shear wall buildings [3]. It relates the design displacements of the SFRS at the top of the building to the profile of deformation demands over the height of the building. Fig. 1 is the envelope of interstory drift ratios as a ratio of the global drift ratio \( \Delta/h_w \), where \( \Delta \) is the design lateral deflection at the top of the gravity-load resisting frame, in the principal direction of the frame, and \( h_w \) is the height of the building measured to the base of the plastic hinge zone in the SFRS. A gravity-load resisting frame may consist of a single column or bearing wall interconnected to a shear wall by slabs (and beams); or may consist of many interconnected columns and/or walls. There are usually several different gravity-load resisting frames in each direction of a building.

![Figure 1 - Envelope of minimum interstorey drift ratios over building height.](image)

The interstory drift envelope in Fig. 1 was developed from two separate nonlinear studies. The interstory drifts for the top 75% of the building height come from the results of nonlinear dynamic analysis of shear walls buildings [3]. The simple piece-wise linear envelope is very similar to the mean interstory drift envelopes for cantilever shear walls of varying height. The top part of the envelope comes from the first-mode response of the buildings, while the middle portion of the envelope comes from the higher modes. The large interstory drift ratio at the base of the wall is due to shear strains that occur in the plastic hinge region of shear walls [5], [6].

There are a number of different ways that the envelope in Fig. 1 can be used to determine the demands on the gravity-load resisting frame. When the gravity frame has unique features like a large transfer girder at one or perhaps a few levels, a computer model of a small portion of the frame can be developed and then subjected to displacements that result in the interstory drift given in Fig. 1 for the height of the floor level. It is also possible to develop a computer model of the entire gravity frame and subject it to a single displacement profile that results in the envelope of interstory drifts at all levels. That displacement profile is shown as a solid line in Fig. 2. The expression for the nonlinear displacement profile over the lower 75% of the building height is given in the figure, while the displacement profile for the top 25% of height varies linearly (uniform interstory drift).
The displacement profile that results in the envelope of interstory drifts simultaneously over the height of the building is not expected to occur during an earthquake as it causes a top displacement equal to 1.26 times the design lateral deflection at the top of the gravity-load resisting frame $\Delta$. While it is convenient to do one analysis to determine all the demands on the gravity-load resisting frame, it is important to note that the forces calculated in the vertical load transmitting members (columns, bearing walls, transfer girders, etc.) will be over predicted assuming the maximum interstory drift occurs at every level simultaneously. This may not be a significant issue as the results from such an analysis may need to be manually adjusted to account for yielding of the horizontal frame members. This is discussed further under “Estimating Shear Force Demands” below. The two dashed lines in Fig. 2 show two possible displacement profiles that together give the envelope of interstory drifts. The first mode type profile gives the largest interstory drifts near the top, while the higher mode type profile gives the demands in the lower levels.

**Uniform Gravity Frame and Uniform Interstory Drifts**

When the gravity-load resisting frame is uniform over a number of stories, as is often the case, the column drift ratios can be estimated from the interstory drift ratios knowing only the relative stiffnesses of the columns and floor systems. Inherent in this analysis is the assumption that the interstory drift is uniform over a number of stories. Fig. 3 shows the relationship for three cases. Case A is a two-column frame where the interstory drift ratios are due to the relative lateral displacements of the floors. Case A also provides an approximate solution for the end column in most frames. Case B is for an interior column of a multi-column frame where the floor on either side of the column has the same properties. Due to the floor framing in on both sides of the interior column, the column drifts are larger than in Case A. Case C is a single column connected directly to a shear wall. Since the shear wall is assumed to be infinitely rigid, the column demands are increased. Note that when the floor rigidity is more than about five times the

![Fig. 2 – Single displacement profile resulting in the envelope of interstory drifts.](image-url)
column rigidity ($\alpha < 0.2$), e.g., due to a stiff floor beam, the column drifts are larger than the building drifts. This happens because of the deflected shape of the beam – the infinitely rigid wall forces the beam to have the same slope at one end, while at the other end; the flexible column is not able to bend the beam in double curvature. Thus the beam is bent in single curvature and this imposes a reverse slope on the column at the floor levels.

There is an additional magnification of drifts that occurs in the gravity-load resisting frames that include long walls such as shear walls. Due to the portion of the frame made up of solid wall being rigid, column drifts are magnified by an additional factor: $1 + (l_w + l_c)/2l_{flr}$, where $l_w =$ the horizontal length of the wall, $l_c =$ the horizontal dimension of the column and $l_{flr} =$ the clear span of the floor system.

Fig. 3 – Ratio of column-to-building drifts where frame members and building drifts are relatively uniform.

**Plastic Hinge Regions of Shear Wall Buildings**

The floor systems in concrete buildings are often thin flat slabs or plates that are very flexible out of plane. The column drifts due to frame action, given in Fig. 3, is very small when the floors are flexible. However there is a second way that multiple floors cause bending of columns. Flat slabs/plates are very stiff in-plane and will force columns to experience the same lateral displacement profile as the shear walls at the floor levels. For this analysis, thin floor systems can be idealized as axially-rigid members with hinges (pins) at each end. If the interstory drift is constant (building has a linearly varying deflection), the columns will remain perfectly straight and parallel to the shear walls and will not experience any bending (zero column drift). On the other hand, if the shear walls experience significant curvature (the interstory drift changes significantly from one floor to the next), the column will experience bending as a result of being
interconnected to the shear walls by many pin-ended rigid links [7].

The largest curvature demands on shear walls occur in the plastic hinge regions. A simple solution to ensuring the gravity-load columns will have adequate flexibility is to require that the columns have a greater curvature capacity than the curvature demand associated with the inelastic rotational demands on the shear walls [7]. This can be accomplished by limiting the horizontal compression strain depth, $c$, in gravity-load columns, when the columns are subjected to the factored axial load for earthquake design as follows:

$$c \leq \frac{\varepsilon_{cu}}{(2\theta_{id} + 0.004)} \cdot l_w$$  

where $\varepsilon_{cu} =$ the compression strain capacity of concrete (taken as 0.0035 for unconfined concrete), $l_w =$ the horizontal length of the shear walls or coupled walls SFRS parallel to the distance $c$, which controls the plastic hinge length in the shear walls – the relationship between inelastic rotation demand and curvature demand, and $\theta_{id} =$ the inelastic rotational demand on the SFRS. Simplified procedures are given in CSA A23.3 [2] for estimating $\theta_{id}$ from the design displacement based on principles presented in [8].

**New Detailing Rules for Columns and Walls**

Over the plastic hinge regions, all columns must contain at least buckling-prevention ties (spaced at smaller of six vertical bar diameters or one-half of least dimension of member) and all walls must have tied concentrated vertical reinforcement at each end of the wall and at the intersections of all walls. These requirements are waived if the design displacement is less than $h_w/200$. The concentrated reinforcement in walls must consist of a minimum of four bars and have at least non-seismic column ties, which are spaced at sixteen times the vertical bar diameters or the least dimension of the member. All ties mentioned above shall be detailed as hoops – closed tie with a seismic hook at each end having a minimum 135° bend with six-bar diameter ($\geq 100$ mm) extension anchored in concrete core.

**Estimating Shear Force Demands**

The linear model of the gravity-load resisting frame can be used to determine the additional bending moments, shear forces and axial loads induced the gravity frame members by the seismic deformation demands in Figs. 1 and 2. As the model of the frame is linear, it does not account for hinging of the frame members. Thus the model may predict bending moments that exceed the flexural resistance of the members. As discussed below, the ratio of the induced bending moment to the flexural resistance is used as an indicator of the inelastic flexural deformation demands on frame members.

It is very important to ensure that the gravity frame members will not fail in shear due to the additional shear forces induced in the members because of the seismic deformation demands. If the gravity-load bending moment plus the induced bending moment in a member exceeds the probable flexural resistance of the member (calculated using reinforcement yield strength equal to 1.25 times the specified strength), the shear force needs to be adjusted to that corresponding to the probable flexural resistance of the member. The increased shear force in the horizontal frame
members influence the vertical load that must be transferred from the upper floors of the building by the columns, bearing walls, transfer girders and transfer slabs. Again, the vertical load in these members may need to be adjusted depending on how the induced bending moments in the horizontal members compare with the probable flexural resistances.

**Gravity-Load Resisting Columns and Bearing Walls**

The columns and bearing walls that transfer the gravity loads and additional vertical loads due to seismic deformation demands are perhaps the most important members in the gravity-load resisting frame. Failure of a few of these members can easily result in a partial or complete collapse of the structure. The new requirements include restrictions on thin columns and bearing walls and new limits on the flexural deformation demands on all columns and bearing walls.

**New Axial Load Restrictions on Thin Columns and Bearing Walls**

Recent experimental research [9] has shown that thin columns and walls have much less toughness than regular square tied columns. Once these members reach a certain level of damage, they lose all vertical load carrying capacity. In the same way that spirally-reinforced columns are given a higher maximum axial load resistance $P_{r,max}$ as a ratio of the factored axial load resistance at zero eccentricity $P_{r,0}$ to reflect their enhanced toughness, thin columns and walls now have a lower $P_{r,max}$ to $P_{r,0}$ ratio for gravity load design as shown in Fig. 4. Tied columns need to have a minimum dimension not less than 300 mm in order to qualify for the traditional $P_{r,max} = 0.80P_{r,0}$. When the minimum dimension of a tied column is 200 mm, which is the practical lower limit, $P_{r,max} = 0.60P_{r,0}$. Walls need to have column ties over the full length in order to qualify for the same $P_{r,max}$ as columns. Regular walls (without column ties along the full length) are given a $0.05P_{r,0}$ reduction on $P_{r,max}$ and thus have an upper limit of $P_{r,max} = 0.75P_{r,0}$. In Canada, spirally reinforced columns now have $P_{r,max} = 0.90P_{r,0}$.

![Fig. 4 – Maximum axial load resistance $P_{r,max}$ for gravity load design of thin columns and walls.](image-url)
The reduced $P_{r,max}$ for gravity load design of thin columns and walls will generally reduce the axial load allowed in a given size member. The reduction will depend on the member slenderness and how the designer accounts for slenderness. An additional restriction has been placed on thin walls so that if the interstorey drift ratio determined from an analysis with torsion and including accidental torsion, exceeds 0.5% at any point in the structure, all bearing walls that are used to support gravity loads must contain a minimum of two layers of uniformly distributed reinforcement and the two layers must have a minimum clear spacing of 50 mm. The maximum interstorey drift in is used as an indicator of seismic demands, as well as flexibility of the structure. Walls with a single layer of reinforcement may not be able to tolerate cycles of combined in-plane and out-of-plane displacement.

**Flexural Deformation Capacities of Column and Bearing Walls**

The new seismic design requirements for columns and walls that are part of the gravity-load resisting frame are intended to be a function of the inelastic flexural deformation demands on the member. As the seismic demands on the gravity-load resisting frame are determined using a linear model, the requirements are a function of how much the calculated induced bending moment due to the seismic deformation demands exceeds the factored bending resistance of the member. Factored resistances are used to account for the uncertainty in displacement demands – the resistances are reduced rather than the displacement demands increased. Multiples of factored resistance are used as indicators of inelastic displacement demands. The induced bending moment determined from a linear analysis is limited depending on the type of member, axis of bending in walls, and level of applied axial compression as given in Table 1.

**Table 1 – Maximum induced bending moments in gravity-load columns and bearing walls determined from a linear analysis.**

<table>
<thead>
<tr>
<th>Type of column or bearing wall</th>
<th>Axial compression $^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_s \leq 0.2 f_c' A_g$</td>
</tr>
<tr>
<td>Columns satisfying requirements for ductile MRF</td>
<td>5.0$M_r$</td>
</tr>
<tr>
<td>Columns satisfying req. for moderately ductile MRF</td>
<td>3.0$M_r$</td>
</tr>
<tr>
<td>Columns with min. dim. $\geq 250$ mm and $\geq 0.4$ max. dim.</td>
<td>2.0$M_r$</td>
</tr>
<tr>
<td>Other columns or walls tied as column over full length</td>
<td>1.5$M_r$</td>
</tr>
<tr>
<td>Strong-axis bending of walls with two layers of reinf. and concentrated reinf. at the ends</td>
<td>1.2$M_r$ $^{(2)}$</td>
</tr>
<tr>
<td>Strong-axis and weak-axis bending of walls with two layers of reinforcement</td>
<td>1.0$M_r$ $^{(2)}$</td>
</tr>
<tr>
<td>Strong-axis and weak-axis bending of walls with a single layer of reinforcement</td>
<td>0.7$M_r$ $^{(2)}$</td>
</tr>
</tbody>
</table>

$^{(1)}$ Linear interpolation to be used for intermediate levels of axial compression; $P_s$ = axial force from factored dead load plus factored live load using earthquake load factors.

$^{(2)}$ The induced bending moment must be determined using $E_c I_c = 1.0 E_i I_g$ for these members.
Design of Gravity-Load Resisting Beams

The process used for gravity-load resisting beams is to adjust the detailing requirements based on the expected level of inelastic demands. As the deformation demands are determined using a linear model of the frame, the requirements are again related to how much the calculated induced bending moment due to the seismic deformation demands exceeds the factored bending resistance of the member. As with columns, factored resistances are used to account for the uncertainty in displacement demands. Table 2 summarizes the requirements. When the calculated induced bending moment is more than five times the factored bending resistance, the design must be changed to reduce the induced bending moment or increase the bending resistance of the member and the structure reanalyzed.

Table 2 – Beam detailing requirements as a function of induced bending moment calculated from a linear analysis of the gravity frame.

<table>
<thead>
<tr>
<th>Induced bending moment</th>
<th>Detailing requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1.0Mr</td>
<td>No additional requirements</td>
</tr>
<tr>
<td>≥ 1.0Mr; but &lt; 2.0Mr</td>
<td>Limited ductility</td>
</tr>
<tr>
<td>≥ 2.0Mr; but &lt; 3.0Mr</td>
<td>Moderately ductile</td>
</tr>
<tr>
<td>≥ 3.0Mr; but &lt; 5.0Mr</td>
<td>Ductile</td>
</tr>
<tr>
<td>≥ 5.0Mr</td>
<td>Not permitted – redesign</td>
</tr>
</tbody>
</table>

The following is a summary of the design requirements for the three different types of ductile gravity-load beams. Limited ductility: at least two continuous longitudinal bars provided at both top and bottom, and stirrups spaced at not more than d/2 throughout length of beam; minimum positive moment reinforcement (that normally must extend 150 mm into continuous supports) must be anchored to develop the specified yield strength in tension at face of support, and; reinforcement ratio must not exceed 0.025 at locations of flexural yielding. Moderately ductile: all of above plus for distance d from where flexural yielding may occur, stirrups spaced at smaller of d/4 and 12 times diameter of smallest enclosed longitudinal bar; all top longitudinal reinforcement terminating in a column must extend to far face of column core and be anchored by a standard 90° hook to develop yield strength of reinforcement. Ductile: all of above plus clear span not less than three times the effective depth; shear design must be in accordance with ductile frame members; for distance d on either side of section where flexural yielding may occur, stirrups must be detailed as hoops and seismic crossties and must be spaced at smaller of d/4 and 8 times diameter of smallest enclosed longitudinal bar; lap splices not permitted within distance d from where flexural yielding may occur; transverse hoop reinforcement provided over depth of beam-column joints, spaced at maximum of 8 times diameter of smallest enclosed longitudinal column bar.

Design of Slab-Column Connections for Seismic Drift Demands

CSA A23.3 has had a requirement to check the punching shear capacity of slab-column connections under seismic drifts since the 2004 edition. The provisions are similar to ACI 318. The maximum gravity load two-way shear stresses determined using seismic load combinations
and excluding shear stresses from unbalanced bending moment, must be within a limit that reduces as the interstory drift increases. Until now, designers have used the results of linear dynamic analysis to determine the interstory drift at each floor level. Linear analysis indicates very small interstory drifts near the base of shear walls. Designers must now use Fig. 1 to determine the interstory drift and this will result in considerably more slab shear reinforcement in the lower levels of buildings.

**Conclusions**

Gravity-load resisting frames constructed in Canada, including in regions with very high seismic hazards, are typically much less resilient than typical gravity-load frames constructed in the US. One reason for this is that virtually all buildings in Canada have shear walls as the SFRS, and these were thought to protect the gravity frame from significant deformation demands. Canada has not experienced a large damaging earthquake that has demonstrated the importance of resilient gravity frames. Concerns over the gravity-load resisting frames in Canada have led to the development of significantly new requirements in the 2015 Canadian Building Code and this paper has briefly summarized these new requirements. The “heart” of the new method is an interstory drift envelope for shear wall buildings. This envelope, which accounts for nonlinear flexural and shear deformations of shear walls, is used to determine the deformation demands on gravity frames as a function of the lateral displacement in the direction of the frame, at the top of the building. Linear analysis can be used as a simple way to estimate demands on the frame and the design requirements depend on how much the calculated bending moment exceeds the factored resistance of the member. New restrictions have been added on the use of thin walls and on the axial load capacity of thin columns and walls.

**References**