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Changes to 2015 Canadian Building Code for High-rise Concrete Wall Buildings Following 2010 Maule Earthquake

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Abstract

Compression failures of numerous thin concrete walls in the 2010 Maule Earthquake motivated an experimental investigation that revealed thin concrete walls subjected to uniform compression strain may have much lower compression strain capacity than previously realized, and thin concrete columns and walls may experience very sudden axial compression failures after a small level of damage to the concrete cover because the thin layer of undamaged concrete between the two layers of horizontal reinforcement easily becomes unstable. As thin concrete columns and walls are commonly used structural members in high-rise concrete buildings in Canada, these experimental findings led to extensive changes to the 2015 Canadian Building Code, which are documented in this paper. The new requirements include: (i) The need to check the maximum compression strain demands over the full height of all concrete shear walls used as the seismic-force-resisting system (SFRS) in buildings, whereas this was previously only required at the base of the shear walls where plastic hinging is expected to occur. (ii) New requirements for tied vertical reinforcement at the base of all low ductility SFRS shear walls to protect the compression end of the walls, whereas this was previously only required in shear walls subjected to higher levels of ductility demand. (iii) New significant reductions in the axial load resistance of thin columns and walls (less than 30 cm thick) resisting gravity loads. The reduction is greater for walls without cross-ties (i.e., typical walls) than for columns, which usually contain cross-ties. (iv) A new requirement to account for strong-axis bending of bearing walls, even though such members are defined as members subjected to axial load and weak-axis bending. (v) New requirements that prevent very thin concrete flexural shear walls from being used to resist lateral load due to wind. (vi) Extensive new requirements to prevent failure of thin concrete columns and walls in gravity-load resisting frames due to the seismic deformation demands on the structure. The limit on the induced column drift depends on the type of member (14 different types are defined varying from a column meeting all the requirements for a ductile moment-resisting frame to a thin wall with a single layer of reinforcement). Also, all gravity-load columns and walls must have a curvature capacity greater than the curvature demand in the plastic hinge regions of the SFRS shear walls, and if the interstorey drift ratio exceeds 0.5% at any point in the structure, all walls that are used to support gravity loads must contain a minimum of two layers of reinforcement.

Keywords: building codes; concrete buildings; shear walls crushing of thin walls; seismic design

1. Introduction

Many modern high-rise concrete shear wall buildings were damaged during the M8.8 Maule Earthquake that occurred in February 2010. A common type of damage was compression failure of thin (about 150 mm thick) concrete walls [1]. Compression failure of a thin concrete wall also contributed to the collapse of the Pyne Gould Building in Christchurch New Zealand in 2011 [2]. Following the observations in the Maule Earthquake, an experimental study was undertaken in Canada to investigate the compression failures of thin concrete walls. The results of that investigation revealed that thin concrete walls may have much less compression strain capacity than previously thought, and much less toughness than previously thought.

Thin structural concrete walls and thin (elongated) concrete columns are commonly used in high-rise buildings in Canada. They are widely used as gravity-load resisting members across all regions of Canada, including the seismically active west coast. [3] They are also commonly used as the seismic-force-resisting elements in very tall buildings in regions of lower seismicity such as in the major metropolitan area of Toronto. They are also sometimes used as the seismic-force-resisting elements in buildings up to 30 m high in the highest seismic regions in western Canada.

As a result of the experimental study on thin concrete walls, significant changes were made to the 2015 Canadian Building Code. This paper briefly summarizes the experimental investigation and presents a complete summary of the changes that were made to the 2014 edition of the Canadian Standard for the design of concrete structures CSA A23.3 [4], which is reference by the 2015 National Building Code of Canada. [5]

2. Experimental Investigation

2.1 Phase 1: Small wall element tests

The experimental program consisted of two phases. In the first phase, many small wall elements were subjected to cyclic axial compression. The main parameters were wall thickness (from 140 to 250 mm), number of layers of horizontal wall reinforcement (no horizontal reinforcement, one layer or two layers), clear cover to horizontal reinforcement, whether the vertical reinforcement had any column ties, and the slenderness of the wall elements.

The wall elements were either 61 or 91 cm high. All vertical reinforcing bars had a nominal diameter of 10 mm, and were welded to small steel plates at the top and bottom of the specimens to ensure full development over the full height of the specimens. The horizontal reinforcing bars had a diameter of 10, 15 or 20 mm. In Canada, walls up to 21 cm thick can have a single layer of reinforcement, while many of the 14 cm thick walls that failed in the 2010 Maule Earthquake had two layers of small diameter reinforcing bars. All specimens were cast in wooden forms in the same position as they were tested – vertical bars in vertical position. The 28-day cylinder compression strength of the ready-mix concrete determined from moist-cured/field-cured cylinders was found to be 30/25 MPa. The wall elements were tested between 1 and 6 months after casting. At 6 months, the concrete compression strength had increased to 32/26 MPa based on moist-cured/field-cured cylinders.

The specimens were loaded under pseudo strain control, i.e., the load was increased until the target average strain was reached. The standard protocol involved five cycles to each strain level. The first target strain level was 0.0005 and the subsequent strain targets were 0.00025 higher than the previous one. Thus a specimen loaded using the standard protocol to a maximum strain of 0.0035 was loaded a total of 65 cycles. Some specimens were subjected to different protocols to study the influence of additional cycles at high strain. The wall elements were subjected to concentric axial compression in a universal testing machine. To prevent failure at the top and bottom, confining steel angles grouted around the ends of the elements.

Four displacement transducers were used to measure the average strain over a gauge length of 46 cm on the 61 cm high specimens and 76 cm on the 91 cm high specimens. In three tests the strain was very non-uniform, while for all other tests, the compression strain was kept fairly uniform. The measured vertical strains were used to estimate the compression force resisted by the vertical reinforcement. The net compression force resisted by concrete and net cross sectional area of concrete were used to calculate the maximum concrete compression stress.

When the wall elements were subjected to a significant variation of compression strain, the results were very different than when the elements were subjected to uniform compression strain. Two specimens were identical except for the compression strain gradient. One specimen failed when the maximum compression strain at one end was 0.0023, while the minimum compression strain at the other end was only 0.0011 (average of 0.0017). The second identical specimen failed when subjected to a uniform compression strain of only 0.0013. Two other specimens were also identical except for the variation of compression strain. One of these specimens failed when the compression strain was 0.006 at one end and 0.0015 at the other end (average of 0.0037), while the other failed at a uniform compression strain of 0.0023. The specimens with significant strain variation behaved similarly. The end subjected to higher compression strains had visible damage prior to failure, while the end that was subjected to lower compression strains had no visible damage prior to failure. Failure occurred when the end subjected to lower compression strains became damaged. This behavior explains why concrete subjected to significant strain gradient is able to tolerate larger maximum compression strains – the concrete subjected to lower compression strain stabilizes the concrete subjected to higher compression strain. In all subsequent tests, the compression strains were kept uniform as the specimens were meant to represent small elements of long thin walls.

The failed specimen shown in Fig. 1(a) is a good example of a specimen subjected to uniform compression strain, which failed at a low compression strain. After cycling to a compression strain demand of 0.0010 with no visible damage, the specimen suddenly “exploded” when pushed to a compression strain of 0.0015. The crushing failure was clearly influenced by the location of the horizontal reinforcement in the wall, which is highlighted by a red circle. This failure is reminiscent of failures observed in thin concrete walls in the 2010 Maule Earthquake.

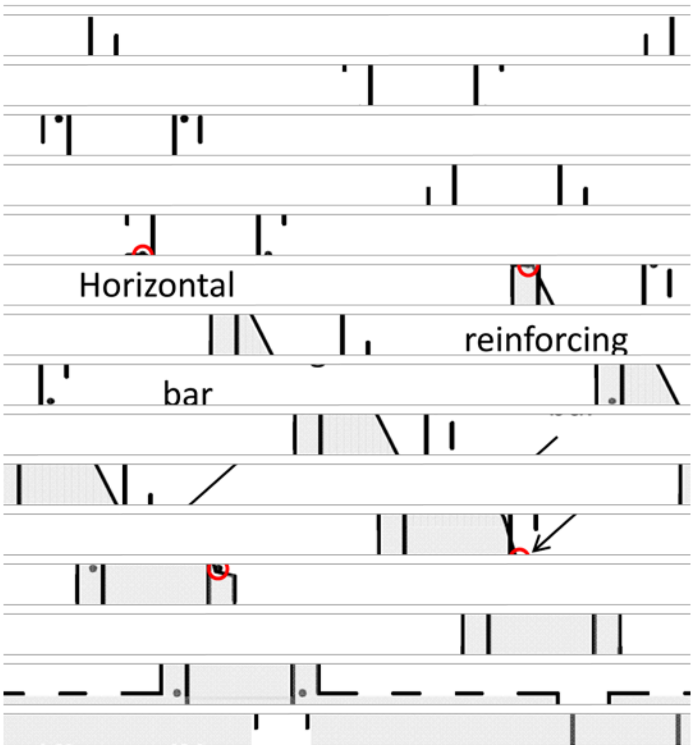

Fig. 1 – Appearance of thin wall elements after crushing failures: (left) photograph of 14 cm thick wall element that failed during the first cycle loading to a maximum compression strain of 0.0015; (right) drawing of failure plane in 20 cm thick column.

A review of all the test results indicates a relatively strong correlation between the height-to-thickness ratio of the wall elements and the minimum measured compression strain capacity of the elements. As the compression stress applied to concrete in the wall approaches the cylinder compression strength, the concrete becomes unstable. The concrete in the more slender wall elements is less influenced by the restraint at the top and bottom of the elements and therefore becomes unstable at lower compression strain values. The wall elements with the lowest (minimum) strain capacity were 140 mm thick, had two layers of 10 mm diameter horizontal reinforcement – similar to many of the walls that failed during the earthquake in Chile – and had a height of 61 cm. Thus the height-to-thickness (*h/t*) ratio of the wall element was 4.35. Specimens with *h/t* = 3.6 had a minimum compression strain capacity of 0.0015, while specimens with *h/t* = 3.0 had a minimum compression strain capacity of 0.0025.

The most important conclusion from the compression tests on wall elements is that concrete walls subjected to uniform compression strain and not containing any tied vertical reinforcement may have a compression strain capacity as low as 0.001. Long concrete walls will have a low strain gradient and thus almost uniform strains. A transverse wall that acts as a compression flange for a shear wall will also be subjected to uniform strain. Additional information about the wall element tests is available elsewhere. [6,7]

2.2 Phase 2: Larger column/wall element tests

In the second phase of the experimental investigation [8,9], a series of five different column/wall elements that varied from a 1:1 square column (40 x 40 cm) to a 1:8 column/wall (14 x 110 cm) were tested under constant axial compression and reverse cyclic lateral load. As the elements were considered to be part of the gravity-load resisting frame in the building (not the seismic-force-resisting system), they were subjected to a high level of axial compression equal to 33% $f_c'A_g$ (1500 kN). The elements where subjected to a reverse cyclic lateral load applied at 141 cm up from the base. As the predicted displacement capacities of the heavily loaded members was relatively small, the target displacements were increased slowly, and the specimens were subjected to three cycles of lateral load to each target displacement level.

The square (1:1) column and the 1:2 column (27.5 x 55 cm) performed very well, resisting the high level of axial compression to drift levels much larger than predicted. [8] The explanation for the good performance is that even after a large amount of concrete around the outside of the column was damaged from the lateral displacements, the center of the columns had sufficient undamaged concrete that the member could continue to resist the applied compression load. On the other hand, the 1:4 column (20 x 80 cm) and the 1:8 column/wall (14 x 110 cm) did not perform nearly as well. One positive observation was that the failure compression strains at the ends of the members in the 1:4 column and 1:8 column/wall were much higher than observed in the small wall element tests. This was due to two reasons: (i) the members were subjected to a very significant strain gradient (due to the short member dimension), and; (ii) the horizontal reinforcement was bent around the end of the column/wall and acted as cross ties.

The most important observation from the 1:4 column and 1:8 column/wall tests was that the members suddenly lost all axial load capacity (a very brittle failure mode) once a portion of the concrete around the perimeter of the member was damaged. The reason was that the undamaged concrete within the vertical reinforcement was very thin and therefore very unstable. The failure mode of these two column/wall tests looked very similar to the failure modes in the brittle small wall element tests. Fig. 1(right) shows the failure plane in the 20 cm thick member. Note that the failure plane was clearly influenced by the location of the horizontal reinforcement.

The low compression strain capacity of thin concrete walls subjected to uniform compression strain (observed in the small wall element tests, and the very sudden axial compression failures of the thin concrete columns and walls (observed in both the small wall element tests and the larger column/wall tests) resulted in significant changes being proposed [10] for the 2015 Canadian Building Code. The changes that were finally made to the Canadian Building Code are described in detail here.

3. Seismic Design Requirements for Concrete Walls in Canada

There are three types of concrete wall seismic-force-resisting systems in Canada. Ductile concrete wall systems are used in high-rise buildings on the seismically active west coast of Canada. Moderately ductile concrete wall systems, which have somewhat fewer design requirements compared to ductile systems, are commonly used in regions of moderate seismicity such as Ottawa and Montreal. The third system, which has traditionally had very few design requirements, are the low ductility system called "conventional shear walls." Conventional shear walls are commonly used in regions of low seismicity such as Toronto, and are permitted in buildings up to 30 m high in regions of high seismicity.

In the description of the new design requirements below, reference will be made to *minimum ties* on vertical reinforcement in the ends of walls. *Minimum ties* consist of closed-ties with seismic hooks that have at least a 135° bend and a six-bar-diameter (but not less than 100 mm) extension into the confined core. The spacing and arrangement of *minimum ties* are similar to gravity-load columns. The maximum spacing is 16 times the smallest vertical reinforcing bar; but not less than the least dimension of the member. The ties must be arranged so that every corner and alternate vertical bar has lateral support provided by the corner of a tie and no vertical bar shall be farther than 150 mm from such a laterally supported bar.

In Canada, the factored compression strength of concrete is taken as 65% of the specified compression strength for all calculations – design for gravity loads and design for earthquake demands. While one reduction factor on the concrete compression strength is not appropriate for both gravity-load design and designing for life safety performance of a structure subjected to a 2% in 50-year hazard, one reduction factor is used for simplicity. Thus the seismic design requirements are adjusted to account for the low estimate of concrete compression strength. This explains why the strain depth limits given below seem rather large.

3.1 Ductile and moderately ductile wall systems

In Canada, ductile and moderately ductile walls must have tied vertical reinforcement at the ends of the walls over the full height of the building. Thus the new test results have not significantly impacted the design requirements for these members. The requirement to explicitly check that the maximum compression strain demand is less than the compression strain capacity of concrete has, until now, only been required in the plastic hinge region at the base of cantilever walls. [11]

Historically it was thought that the inelastic demands on a cantilever shear walls will be concentrated entirely in the plastic hinge region at the base of the walls. In recent years, it has become known that higher modes may apply significant bending moment demands at the mid-height of the wall. As a result of a combination of the new information on the increased strain demands at the mid-height of cantilever walls, and the new information on the reduced strain capacity of thin walls, new requirements have been added that requires designers to confirm that the compression strain demand is less than the compression strain capacity of unconfined concrete. This is accomplished by requiring that the distance from the extreme compression fibre to the neutral axis, c, determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain of 0.0035 at the extreme compression fibre, does not exceed 50% of the wall length for moderately ductile walls. For ductile walls, the limit is reduced to 40% of the wall length. (Clause 21.5.7.1.1 of CSA A23.3-2014)

When concrete walls are arranged into a core around the stairway and elevator shafts, the walls are usually subjected to very low compression strain demands because the transverse wall acts as a wide "compression flange" on the ends of the walls, which reduces the compression strain depth usually to about 10% of the wall length. In these cases, the new requirements do not have any influence on the design of ductile and moderately ductile walls.

3.2 Low ductility wall systems

The results from the experimental study has had a significant impact on the design of low ductility "conventional" shear walls in Canada. As these walls were generally not expected to be subjected to large inelastic curvature demands, they have not had any requirements for minimum tied vertical reinforcement at the ends of the walls, and it was not necessary for designers to check the maximum compression strain demands in the walls. As a result of the recent results on thin concrete walls, additional requirements have been added both at the base of the wall, where yielding of the vertical reinforcement may occur (called the 'potential plastic hinge region'), as well as over the complete height of the wall.

Over the height of the *potential plastic hinge region* (equal to the wall length), concentrated vertical reinforcement, consisting of a minimum of four bars placed in at least two layers, must now be provided at the ends of all walls. This reinforcement must have the minimum column ties described above. With this new requirement, minimum tied vertical reinforcement at the ends of the wall must now be provided near the base of all concrete walls used to resist seismic forces in Canada.

In addition, the designer must confirm that the distance from the extreme compression fibre to the neutral axis, c, determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain of 0.0035 at the extreme compression fibre, does not exceed 50% of the wall length. (Clause 21.6.3.7 of CSA A23.3-2014). At every section above the potential plastic hinge region, the distance from the extreme compression fibre to the neutral axis, c, determined as described above, must not exceed 60% of the wall length. If the distance from the extreme compression fibre to the neutral axis, c, exceeds 30% of the wall length, the minimum concentrated vertical reinforcement (minimum four bars placed in at least two layers with the minimum ties) must be provided at the ends of all walls. (Clause 21.6.3.6 of CSA A23.3-2014)

4. Design of Thin Concrete Walls for Gravity and Wind Loads

The new results on the limited compression strain capacity of thin concrete walls and reduced toughness of thin concrete walls has also resulted in significant changes to the design of concrete walls for gravity loads and wind loads.

4.1 Reduced axial load resistance of thin concrete walls

Traditionally, spirally-reinforced columns have been given a higher *maximum axial load resistance* $P_{r,max}$ as a ratio of the *factored axial load resistance at zero eccentricity*, P_{ro}, to reflect their enhanced toughness. $P_{r,max}$ is the limit on the maximum calculated factored axial compression load that can be applied to a column if there is no applied bending moment, while P_{ro} is the calculated factored axial load resistance with zero bending moment. In North America, the ratio is limited to 0.85 for spirally reinforced columns and 0.80 for tied columns.

The experimental study summarized above 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. Thus thin columns and walls in Canada now have a lower $P_{r,max}$ to P_{ro} ratio for gravity load design as shown in Fig. 2. Tied columns need to have a minimum dimension not less than 300 mm in order to qualify for the traditional $P_{r,max} = 0.80P_{ro}$. When the minimum dimension of a tied column is 200 mm, which is the practical lower limit, $P_{r,max} = 0.60P_{ro}$. 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 $0.05P_{ro}$ reduction on $P_{r,max}$ and thus have an upper limit of $P_{r,max} = 0.75P_{ro}$ (Clause 10.10.4 of CSA A23.3-2014).

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. This is discussed further in Section 4.3.

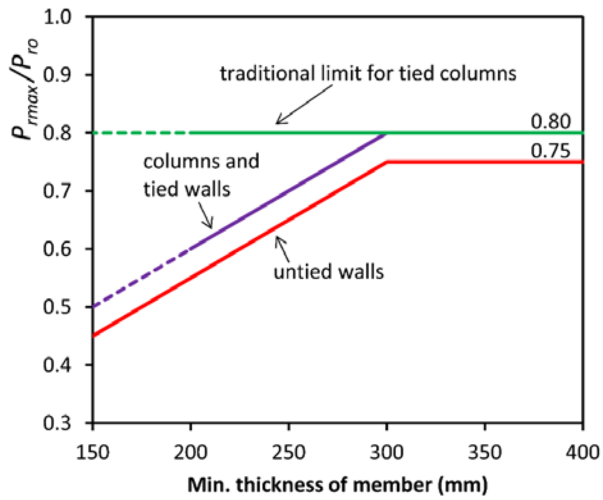

Fig. 2 – Maximum axial load resistance as a ratio of factored axial load resistance at zero eccentricity for gravity load design of thin columns and walls in the 2015 Canadian Building Code.

4.2 Unintended strong-axis bending of bearing walls

In the Canadian building code, *bearing walls* are defined as a wall that supports axial compression, weak-axis bending moment about a horizontal axis in the plane of the wall and the shear forces necessary to equilibrate the bending moments. It has been a relatively common design approach in Canada to designate certain walls in a building as bearing walls and then completely ignore the influence of strong-axis bending. In some cases, the walls that have been designated as the shear walls (resist strong-axis bending) may not be as long as the thin walls that have been designated as bearing walls.

To avoid crushing at the ends of thin bearing walls, the Canadian Building Code now requires that the calculation of factored axial load resistance account for unintended strong-axis bending moments applied to bearing walls. The note to the new clause explains that strong axis bending moments may be applied to bearing walls due to the resultant of the axial load not being at the centroid of the section due to in-plane offset of the wall or may be induced by deformation of the lateral force resisting system subjected to lateral loads such as wind and seismic loads. (Clause 14.2.3 of CSA A23.3-2014)

4.3 Flexural shear walls resisting wind loading

Concern about the reduced compression strain capacity of thin walls has resulted in changes to the procedures for designing flexural shear walls that resist wind loading. Currently, the requirements for wall slenderness allows a designer to justify very thin shear walls to resist wind loading. The new requirements mandate a simpler set of procedures that will not permit such very thin walls thereby eliminating the concern about premature crushing of the flexural compression end of the wall. The procedure depends on whether the shear wall is subject to a low level or high level of axial compression.

4.3.1 Low level of axial compression

When the axial compression applied to a wall is low enough that the vertical reinforcement at one end of the wall will yield in tension before the maximum compression strain at the other end of the wall reaches 0.0035, the compression end of the wall shall have a wall thickness of at least 5% (1/20) of the unsupported height of the wall, unless the compression strain depth, *c*, is smaller than the lesser of 4 times the wall thickness or 30% of the wall length; or a continuous line of lateral support is provided to the compression end of the wall by a cross wall or wall flange. (Clause 14.4.2.1 of CSA A23.3-2014)

4.3.2 High level of axial compression

When the axial compression force applied to a flexural shear wall is such that the tension reinforcement will not yield, design of the wall shall consider additional slenderness requirements. These additional slenderness requirements need not apply to any part of a wall that lies within a distance of 3 times the wall thickness from a continuous line of lateral support provided by a cross wall or a flange.

The Canadian Building Code permits the resistance (accounting for slenderness) of a wall subjected to the axial compression and bending moment about the strong axis (Fig. 3 – left) to be determined using a reduced length of wall subjected to a concentric axial compression, that is statically equivalent to the axial load and bending moment, applied to the full length of wall (Fig. 3 – right). This concept is illustrated in Fig. 3. The reduced length of wall must satisfy the normal procedures used to account for slenderness in concrete columns. (Clause 14.4.2.2 of CSA A23.3-2014)

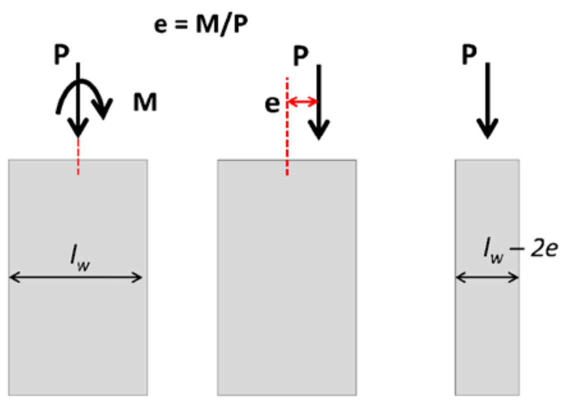

Fig. 3 – Procedure used to account for slenderness of compression zone in flexural shear walls subjected to combined axial compression and strong axis bending such that the flexural tension reinforcement does not yield

5. Gravity-load resisting walls subjected to seismic deformation demands

In Canada, thin concrete walls are commonly used as the vertical members (equivalent to "columns") in gravity-load frames. 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 resisting columns. The Canadian Building Code, like most other building codes, required 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. These requirements have been significantly enhanced for thin columns and walls (Clause 21.11 of CSA A23.3-2014).

5.1 Analysis of gravity-load resisting frame

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.

Most Canadian designers do not use nonlinear analysis for the seismic 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 [12]. It relates the design displacements of the SFRS at the top of the building to the envelope of interstorey drift ratios over the height of the building. For example, the maximum interstorey drift ratio is assumed to be 1.6 times the global drift demand Δ/h_w, and the minimum interstorey drift ratio at the base of the wall is assumed to be 0.7 Δ/h_w (Clause 21.11.2.2 of CSA A23.3-2014).

There are a number of different ways that the envelope of interstorey drift ratios 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 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 interstorey drift ratios for the height of the floor level. On the other hand, 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 interstorey drift ratios knowing only the relative stiffnesses of the columns and floor systems. [13]

The floor systems in Canadian concrete buildings are often thin flat slabs or flat plates that are very flexible out of plane. The column drifts due to frame action (discussed above) 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 interstorey 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 interstorey 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.

The largest curvature demands on shear walls occur in the plastic hinge regions of the walls. 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, and this has been adopted as a new requirement in the Canadian Building Code (Clause 21.11.3.3.2 of CSA A23.3-2014).

5.2 Flexural deformation capacities of columns 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. When the seismic demands on the gravity-load resisting frame are determined using a linear model, as they usually are in Canada, 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 (Clause 21.11.3.3.3 of CSA A23.3-2014).

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 (Clause 21.11.3.3.1 of CSA A23.3-2014).

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

Type of column or bearing wall	Axial compression (1)	
	$P_s \le 0.2 f_c' A_g$	$P_s \ge 0.4 f_c' A_g$
Columns satisfying requirements for *ductile* MRF	$5.0M_r$	$3.0M_r$
Columns satisfying req. for *moderately ductile* MRF	$3.0M_r$	$2.0M_r$
Columns with min. dim. ≥ 250 mm and ≥ 0.4 max. dim.	$2.0M_r$	$1.5M_r$
Other columns or walls tied as column over full length	$1.5M_r$	$1.0M_r$
Strong-axis bending of walls with two layers of reinf. and concentrated reinf. at the ends	$1.2M_r$ (2)	$0.8M_r$ (2)
Strong-axis and weak-axis bending of walls with two layers of reinforcement	$1.0M_r$ (2)	$0.7M_r$ (2)
Strong-axis and weak-axis bending of walls with a single layer of reinforcement	$0.7M_r$ (2)	$0.5M_r$ (2)

(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 $E_c I_e = 1.0 E_c I_g$ for these members.

6. Conclusions

The numerous observed compression failures of thin concrete walls in modern high-rise concrete shear wall buildings during the 2010 M8.8 Maule Earthquake resulted in considerable concern being raised in Canada where thin concrete walls and thin concrete columns are commonly used as important gravity-load resisting members in high-rise buildings. These members are also sometimes used in Canada as the primary seismic-force-resisting members in high-rise buildings. As a result of the concern, a simple two-phase experimental program was conducted on thin concrete walls and thin concrete columns. The results of these tests have been partially reported previously and thus are only briefly summarized in the current paper.

The main conclusions from the experimental investigation is that thin concrete walls subjected to uniform compression strain may have much lower compression strain capacity than previously realized, and thin concrete columns (with cross-ties) and thin concrete walls (without cross ties) may experience very sudden axial compression failures after a small level of damage of the cover concrete due to the thin layer of undamaged concrete between the two layers of horizontal reinforcement.

The conclusions from the experimental investigation led to very significant changes being made to the Canadian Building Code, specifically to the 2014 edition of the Canadian Standard for the design of concrete structures CSA A23.3 [4], which is reference by the 2015 National Building Code of Canada. [5] These changes can be summarized as follows:

Moderately ductile and ductile seismic-force-resisting shear walls: designers must now confirm that the compression strain demand on SFRS shear walls is less than the compression strain capacity of unconfined concrete by ensuring the compression strain “depth” c (distance from extreme compression fibre to neutral axis, determined by plane sections analysis for factored axial load acting on wall and bending moment causing the maximum compression strain of 0.0035) does not exceed 50% of the wall length for moderately ductile walls and 40% of the wall length for ductile walls.

Low ductility (conventional) seismic-force-resisting shear walls: at base of all walls, tied vertical reinforcement must now be provided at the ends of all shear walls. Also, designers must confirm that the compression strain “depth” c does not exceed 50% of the wall length. At every section above the base, the compression strain “depth” *c* must not exceed 60% of the wall length. If compression strain “depth” *c* exceeds 30% of wall length, minimum concentrated vertical reinforcement must be provided at the ends of all walls.

Thin concrete members resisting gravity loads: columns that are less than 30 cm thick now have a reduced axial load resistance. Tied columns that are 20 cm thick have a 25% reduction in axial load resistance. Columns that are between 20 and 30 cm thick have a proportionally reduced axial load resistance. All walls have a reduced axial load resistance. Walls that are 30 cm thick or more have a 6.25% reduction in axial resistance, while walls that are 20 cm thick have a 31% reduction in axial load resistance. Walls that are between 20 and 30 cm thick have a proportionally reduced axial load resistance.

Bearing walls (defined as subjected to axial load and weak-axis bending): the calculation of factored axial load resistance of bearing walls must now account for unintended strong-axis bending moments due to the resultant of the axial load not being at the centroid of the section or due to deformation of the lateral force resisting system subjected to lateral loads from wind or seismic demands.

Flexural shear walls resisting lateral load due to wind: when the axial compression applied to a wall is low enough that the vertical reinforcement at one end of the wall will yield in tension before the maximum compression strain at the other end of the wall reaches the limit for unconfined concrete, the compression end of the wall shall have a thickness of at least 1/20th of the unsupported height, unless the compression strain “depth,” *c*, is less than 30% of the wall length. When the axial compression force applied to a flexural shear wall is large enough that the tension reinforcement will not yield, new slenderness requirements must be satisfied for the flexural compression zone of the wall. These new requirements will ensure that very thin concrete walls are not used to resist large compression demands.

*Gravity-load columns and walls subjected to seismic deformation demand*s: new requirements must be satisfied to ensure all structural members not designated as a part of the SFRS are able to support the gravity loads while being subjected to the design seismic displacements. A new simplified analysis procedure is prescribed for gravity-load resisting frames in shear wall buildings. When the 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 exceeds the factored bending resistance of the member. The maximum induced bending moment depends on the type of member, axis of bending in walls, and level of applied axial compression. The highest limit is 5.0 times the factored bending resistance for columns subjected to a low level of axial compression and satisfying all design requirements for ductile moment-resisting frames. The lowest is 0.5 times the factored bending resistance for walls with a single layer of reinforcement. The 12 other cases lie between these two limits. In addition, all gravity-load columns must have a greater curvature capacity than the curvature demand associated with the inelastic rotational demands in the plastic hinge region of the shear walls. Finally, if the interstorey drift ratio exceeds 0.5% at any point in the structure, all walls that are used to support gravity loads must contain a minimum of two layers of reinforcement.

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8. References

[1] Sherstobitoff J, Cajiao P, Adebar P (2012): Repair of an 18-story shear wall building damaged in the 2010 Chile earthquake. *Earthquake Spectra*, **28** (S1), 335-348.

[2] Beca Carter Hollings & Ferner Ltd. (2011): Investigation into the collapse of the Pyne Gould Corporation Building on 22nd February 2011. *Report*, 46 pp.

[3] Adebar P, Bazargani P, Mutrie J, Mitchell D (2010): Safety of gravity-load columns in shear wall buildings designed to Canadian standard CSA A23.3. *Can. J. Civ. Eng.*, **37** (11), 1451–1461.

[4] CSA (2010): *Design of concrete structures. Standard CSA-A23.3–04*, Canadian Stand. Association, Mississauga, Ont.

[5] NRCC (2015): *National Building Code of Canada*. Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

[6] Lorzadeh A (2012): Experimental study to investigate compression failures of thin concrete walls. *M.A.Sc. thesis*, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, 444 pp.

[7] Adebar P, Lorzadeh A (2012): Compression failure of concrete walls. *Proc. of 15th World Conf. on Earthquake Engineering*, Lisbon, Portugal.

[8] Chin H (2012): Bending displacement capacity of elongated reinforced concrete columns. *M.A.Sc. thesis*, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, 441 pp.

[9] Adebar P, Barzargani P, Chin H (2012): Design of gravity-load columns in shear wall buildings. *Proc. of 15th World Conf. on Earthquake Eng*., Lisbon, Spain, 2012.

[10] Adebar P (2013): Compression failure of thin concrete walls during 2010 Chile earthquake: lessons for Canadian design practice. *Can. J. Civ. Eng.*, **40** (8), 711-721.

[11] Adebar P, Mutrie J, DeVall R (2005): Ductility of concrete walls: the Canadian seismic design provision 1984 to 2004. *Can. J. Civ. Eng.*, **32** (6), 1124-1137.

[12] Dezhdar E (2012): Estimating flexural demands in high-rise cantilever shear walls. *Ph.D. thesis*, Dept. of Civil Eng., University of British Columbia, Vancouver, BC, Canada.

[13] Adebar P, DeVall R, Mutrie J (2014): Design of Gravity-load resisting frames for seismic displacement demands. *Proc. of 10th US Nat. Conf. on Earthquake Eng.*, Anchorage, Alaska, 10 pp.