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SEISMIC DESIGN OF CONCRETE SHEAR WALL BUILDINGS: NEW REQUIREMENTS OF CSA STANDARD A23.3–2014

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ABSTRACT: Recent research, much of it done as part of the Canadian Seismic Research Network, has resulted in major changes to the seismic design requirements for concrete shear wall buildings in the 2014 edition of CSA A23.3. Effective stiffness of a shear wall now depends on the ratio of elastic bending moment demand to flexural strength. Minimum height of plastic hinge regions depends on building height. Flexural yielding above the plastic hinge regions is deemed acceptable as curvature demands are known to be small; additional ductility requirements ensure flexural compression failure will not precede yielding of vertical reinforcement above the plastic hinge regions. A lower-bound shear correction factor is introduced to compensate for the short duration of the maximum higher mode shear force pulse, lack of co-occurrence with maximum bending strains and shear ductility of walls. The large shear force reversals that occur because of stiff diaphragms below the plastic hinge regions must be accounted for. There are new minimum anchorage requirements for horizontal steel in walls, revised requirements for the design of squat walls and for the design of moderately ductile coupled shear walls. The largest changes relate to the design of gravity-load resisting frames for seismic displacements and the seismic design of building foundations accounting for foundation movements.

1. Introduction

The seismic design of concrete buildings in Canada is done in accordance with the *National Building Code of Canada* (NBCC) and Canadian Standard CSA A23.3 *Design of Concrete Structures* (CSA A23.3), which is referenced by NBCC. NBCC defines seismic loads and analysis procedures, while CSA A23.3 defines design and detailing requirements for standard concrete seismic force-resisting systems.

The 2014 edition of CSA A23.3 contains many new seismic design requirements, particularly for the design of shear wall buildings. Many of these new requirements are the result of knowledge gained from research conducted as part of the Canadian Seismic Research Network (CSRN). This paper describes some of these new seismic design requirements in CSA A23.3–2014.

2. Seismic Force-Resisting Systems for Shear Wall Buildings

Table 1 summarizes the concrete shear wall seismic force-resisting systems (SFRS) according to 2015 NBCC. Coupled walls are categorized separately because of the increased ductility and overstrength in these systems due to the yielding that is required in the coupling beams before an inelastic mechanism can occur. Generally, SFRS can be designed to different levels of ductility. *Conventional* systems have the fewest seismic design requirements, the lowest level of ductility and the lowest force reduction factors; while *Ductile* systems have the most stringent design requirements and the highest level of ductility. *Moderately Ductile* is the intermediate system. Individual shear walls can be slender flexural (cantilever) walls or can be squat walls. The latter cannot be designed as a ductile system. The majority of the recent changes to CSA A23.3 are in the shear wall design requirements. *Moderately Ductile* coupled shear walls are an entirely new system for 2014.

Table 1 – Summary of concrete shear wall seismic force-resisting systems (SFRS).

Type of SFRS	R_d (1)	R_o (2)	Height limit (m) Highest Seismic Region
Coupled shear walls			
Ductile – fully coupled	4.0	1.7	None
Ductile – partially coupled	3.5	1.7	None
Moderately Ductile – fully coupled	2.5	1.4	60
Moderately Ductile – partially coupled	2.0	1.4	60
Shear walls			
Ductile	3.5	1.6	None
Moderately Ductile	2.0	1.4	60
Conventional	1.5	1.3	30
Other (undefined) systems	1.0	1.0	Not Permitted

(1) R_d = Ductility-based force reduction factor;
(2) R_o = Overstrength-based force reduction factor.

3. Design of Concrete Shear Wall Systems

The level of ductility that is typically used for shear wall systems in Canada depends primarily on the seismicity of the region. *Ductile* shear walls are generally the system of choice for tall buildings on the west coast where seismic demands are highest (spectral displacement S_d = 200 mm at a period T = 2 s based on 2010 NBCC); while *Moderately Ductile* shear walls are typically used in Ottawa and Montréal where seismic demands are lower (S_d = 40/50 mm at T = 2 s). *Conventional* shear walls are typically used in the Toronto area where seismic demands are lower still (S_d = 15 mm at T = 2 s).

The shear wall design requirements that have existed in CSA A23.3 were developed primarily by designers from the west coast of Canada. As a result, the requirements were focused on ductile shear walls. One significant change for 2014, is that CSA A23.3 now “spells out” all requirements for *Moderately Ductile* shear walls. Also, as conventional shear walls are permitted to a height of 30 m (10 or 11 stories) in the highest seismic regions (see Table 1), additional requirements have been added for these systems.

3.1. Effective Stiffness of Concrete Shear Walls

As many of the requirements in CSA A23.3 are displacement based, it is important to make a good estimate of the expected maximum displacement due to the design earthquake. Similar to other codes, CSA A23.3 specifies that for the purpose of determining deflections of a structure, reduced section properties shall be used to account for concrete cracking. The previous versions of CSA A23.3 gave the effective moment of inertia of concrete shear walls as a function of the axial compression stress applied to the wall as proposed by Paulay (1986) and reaffirmed in an experimental study (Adebar, Ibrahim and Bryson, 2007). The effective stiffness was defined as the slope of an equivalent linear load-deformation relationship.

The maximum displacement of a building depends on the slope of the load-deformation relationship; but depends even more on the amount of hysteretic damping. Axial compression on a wall increases the bending moment to cause cracking and thus increases the linear range; but reduces the hysteretic damping because the axial compression closes the flexural cracks when the lateral load is removed. A concrete wall with a large amount of vertical reinforcement and no axial compression will have an almost elastic-plastic response, while a concrete wall with high axial compression and very little vertical reinforcement will have an a nonlinear elastic response similar to rocking. For the same strength and initial stiffness, a wall with higher axial compression will actually have a lower effective stiffness even though the load-deformation response will be more linear. Based on the work of Adebar and Dezhdar (2015b), the following relationship is specified for the effective moment of inertia of a concrete shear wall:

$$I_e/I_g = 1.0 - 0.35\left(R_d R_o/\gamma_w - 1.0\right) \geq 0.5 \tag{1}$$

where the wall overstrength γ_w can be taken equal to R_o for simplicity so that the effective moment of inertia depends only on R_d as given in Table 1: $I_e = 0.50I_g$ for ductile walls, $I_e = 0.65I_g$ for moderately ductile walls and $I_e = 0.83I_g$ for conventional walls. As the current provisions typically give $I_e = 0.70I_g$ for ductile walls, the displacement demands will increase for these walls.

3.2. Design Bending Moment and Shear Force Envelopes

Linear dynamic (response spectrum) analysis (RSA) is normally used to determine the design shear force and bending moment envelopes for shear wall buildings. When first yielding of vertical reinforcement is expected to occur at a well-defined critical section near the base of the wall, and there are no irregularities that will cause yielding, the walls are designed for a single plastic hinge region at the base. The properties of the wall cross section that affect the bending resistance of the wall, such as concrete geometry, concrete strength and amount of reinforcing steel, must be maintained over the height of the plastic hinge region. To reduce the amount of yielding that will occur above the plastic hinge region, the factored bending moments at elevations above the plastic hinge region are increased by the ratio of factored bending moment resistance to factored bending moment, both calculated at the top of the plastic hinge region.

The minimum height of the plastic hinge region, where special detailing is required to ensure the wall can tolerate significant inelastic rotation, was previously specified as 1.5 times the wall length. Based on the results from Bohl and Adebar (2011), that height has been changed to 0.5 times wall length plus 0.1 times wall height. For a wall with a height-to-length ratio of 10, the new and old requirements are the same, while for a wall with a height-to-length ratio of 5.0, the new requirement is reduced to 1.0 times the wall length (from 1.5).

The factored base shear force determined by RSA must be increased to account for flexural overstrength and the inelastic effects of higher modes; however, the factored shear force does not need to be taken larger than the shear force calculated using $R_d R_o$ equal to 1.3 as this is the required strength of a fully elastic structure with overstrength. To account for flexural overstrength, the factored base shear force for a ductile wall is increased by the ratio of probable bending moment capacity to factored bending moment, both calculated at the base of the wall. For moderately ductile walls, the same calculation is done using nominal bending moment capacity; while for conventional walls, factored bending moment capacity is used.

3.2.1. Influence of Higher Modes on Bending of Walls

Traditionally, the design of cantilever shear walls in Canada was based on the philosophy that a well-detailed plastic hinge at the base of the wall will protect the wall from inelastic demands above the plastic hinge (Paulay, 1986). It is now well-known that due to higher modes, large bending moment and shear force demands will occur above the plastic hinge. It was initially thought that for 2014, ductile detailing would need to be extended over the wall, especially around mid-height where large bending moment reversals are known to occur; or the flexural strength of shear walls would need to be significantly increased to prevent yielding (Boivin and Paultre, 2012). An extensive study of cantilever shear walls (Dezhdar and Adebar, 2015a) has shown that neither additional ductile detailing nor additional flexural strength is required because the curvature demands above the plastic hinge regions are determined to be relatively small as long as the nonlinear analysis is done correctly. This includes selecting and scaling the ground motions in a suitable way for tall shear wall buildings (Dezhdar and Adebar, 2015b), correctly accounting for the large increase in wall flexibility due to flexural cracking (Adebar, Ibrahim, Bryson, 2007), and accounting for shear cracking. To tolerate the curvature demands, concrete shear walls need to have a minimum level of ductility over the full height and this is discussed further in Section 3.3.

3.2.2. Influence of Higher Modes on Shear Force Demands

The second important effect of higher modes is on the design shear force envelope. The factored shear force envelope for each wall is determined from RSA using the same force reduction factors used to determine the bending moment envelope. In an idealized single degree of freedom cantilever with all the mass concentrated at one height, the shear force will always be proportional to the bending moment and cannot increase after a flexural hinge forms at the base; but in a cantilever shear wall building with distributed mass over the height, the shear force is not limited by a flexural hinge and will continue to increase as the intensity of ground shaking increases. No guidance was given in the previous version of CSA A23.3 on how to correct the shear force envelope to account for these effect.

As described in Adebar, Dezhdar and Yathon (2015), recent nonlinear analysis of cantilever shear walls where the ground motions are properly selected and scaled, and the large increase in wall flexibility due to flexural cracking is modeled correctly, have indicated that the shear correction factor should increase approximately linearly from 0 to 2.0 as the ratio of elastic bending moment demand to bending moment capacity increases from 1.0 (elastic wall) to 3.0, and the correction factor should remain constant at 2.0 for larger bending moment ratios (weaker walls). For western Canada, these numbers are suitable for any cantilever shear wall building with a fundamental period equal to or greater than 1.0 s. Based on the work of Boivin and Paultre (2012), these numbers are suitable for buildings with a fundamental period as low as 0.5 s in eastern Canada.

The CSA A23.3 technical committee decided to implement a shear correction factor that did not exceed 1.5. There are a number of reasons for this as explained in detail in Adebar, Dezhdar and Yathon (2015): (i) the maximum shear force occurs during one cycle of loading for a very short instant of time and the peak shear forces in other cycles are considerably smaller; (ii) the maximum shear force typically does not occur when the base hinge rotation is large, and; (iii) nonlinear analysis has demonstrated that shear walls can tolerate yielding of horizontal reinforcement without failure.

3.2.3. Below the Plastic Hinge Region

In most high-rise buildings, the plastic hinge region occurs above the ground level (or podium structure) and the tower shear walls extend through a number of below-grade levels to the foundation. The tower shear walls are inter-connected with long perimeter foundation (retaining) walls by numerous floor diaphragms. If the diaphragms remain uncracked, they will have very high in-plane stiffness, which may cause large shear force reversals in the tower shear walls. A new clause has been added to deal with this complex design case. It requires that the factored shear force and corresponding factored bending moment applied to the shear wall below the plastic hinge be determined using an analysis (Rad and Adebar, 2009) that considers the lower-bound and upper-bound values of effective stiffnesses of the members as appropriate to determine a safe estimate of the factored shear force. In addition, the wall immediately below the critical section at the base, where yielding of the vertical reinforcement initiates, must contain a minimum of 20% additional vertical reinforcement to prevent yielding from spreading to the portion of wall between stiff diaphragms (Rad and Adebar, 2009). The influence of flexural cracking of diaphragms on the in-plane stiffness was recently studied by Mahmoodi and Adebar (2015).

3.3. Ductility of Shear Walls

CSA A23.3 has a rigorous displacement-based method for determining that a concrete shear wall has sufficient flexural ductility (Adebar, Mutrie, DeVall, 2005). Normal Canadian practice is to not provide confinement reinforcement at the ends of shear walls; however, tied vertical reinforcement is provided at the ends of all walls in the plastic hinge regions. The spacing of the ties (six times the diameter of the smallest vertical bar) will prevent buckling of vertical reinforcement; but will not provide a high level of confinement to concrete. Thus the main purpose of the ductility provisions is to ensure that the maximum compression strain demand does not exceed the compression strain capacity of unconfined concrete (assumed to be 0.0035) given the level of axial compression applied to the wall and the geometry of the walls. The ductility requirements are one reason that core walls are preferred for tall buildings in Canada – perpendicular walls provide a compression flange that keeps the compression strain demands small.

While there have been strict ductility requirements in the plastic hinge regions of ductile and moderately ductile walls, there have been essentially no ductility requirements outside the plastic hinge region or in conventional wall systems. As result of the higher mode curvature demands discussed above and the many flexural compression failures observed in thin concrete shear walls during the 2010 Chile (Maule) earthquake (Adebar, 2013), new requirements have been added to ensure a minimum level of ductility in all concrete shear walls over the full building height. The new requirements are summarized in Table 2.

Table 2 – Shear wall ductility requirements in terms of length of wall under flexural compression strain c and overall wall length l_w: new limits shown in red.

Portion of shear wall	Global drift demand Δ/h_w	Type of shear wall: Ductile	Type of shear wall: Moderately ductile	Type of shear wall: Conventional
Above plastic hinge region	-- †	$c \le 0.4\ l_w$	$c \le 0.5\ l_w$	$c \le 0.6\ l_w$‡
Within plastic hinge region	≤ 0.005	$c \le 0.3\ l_w$	$c \le 0.35\ l_w$	$c \le 0.5\ l_w$
	≈ 0.010	$c \le 0.17\ l_w$	$c \le 0.23\ l_w$	
	≈ 0.015	$c \le 0.12\ l_w$	$c \le 0.17\ l_w$	

† Global drift demand does not influence ductility limits above plastic hinge regions;
‡ Requirement for tied vertical reinforcement at ends of wall can be waived if $c \le 0.3\ l_w$.

Above the plastic hinge regions, the required concentrated reinforcement at the ends of walls are tied with seismic hoops spaced at the regular (non-seismic) column tie spacing equal to thickness of wall or sixteen times diameter of smallest vertical bar diameter. This low level of transverse reinforcement helps to prevent premature crushing of concrete (Adebar, 2013).

3.4. Anchorage of Horizontal Reinforcement

CSA A23.3 has traditionally had a requirement that horizontal reinforcement in walls shall extend to the ends of the wall and shall be anchored at each end within the boundary elements or within the regions of concentrated reinforcement. Different designers have interpreted this in different ways. Most designers of *Ductile* shear walls have placed the horizontal reinforcement inside the concentrated reinforcement at the end of the wall and over the plastic hinge regions, have made the region of tied concentrated reinforcement at least as long as the development length of the horizontal reinforcement or have provided a standard 90-deg. hook on the end of the horizontal reinforcement. Designers of *Conventional* shear walls often do not provide any tied vertical reinforcement at the ends of the wall and when they do, anchor the horizontal reinforcement outside the vertical reinforcement in the concrete cover with only a straight bar. In order to ensure adequate anchorage of horizontal reinforcement in lower ductility walls, new minimum anchorage details have been added to CSA A23.3. *Conventional* walls require at least a 90-deg. hook around the end vertical bars; *Moderately Ductile* walls require a U-bar around the end vertical bars with staggered lap splices minimum five times the wall thickness for the end of the wall, and ; *Ductile* walls require the horizontal bars to be anchored inside tied vertical reinforcement. In plastic hinge regions of *Ductile* walls, horizontal bars must be fully developed within the tied vertical reinforcement.

3.5. Squat Shear Walls

The minimum requirements for squat shear walls, with height-to-length ratios less than 2.0, have been relaxed in 2014 for cases where the wall is longer than needed. The requirements for increased minimum reinforcement, anchorage of horizontal reinforcement described above and tied concentrated reinforcement at wall ends are all waived if the applied shear stress is below a certain limit.

The quantity of distributed horizontal and vertical reinforcement required for shear is calculated assuming a uniform diagonal compression stress field with no concrete contribution; but the inclination of the diagonal compression can be freely selected between 30 and 45 deg. to adjust the relative amounts of vertical and horizontal reinforcement. The maximum diagonal compression stress limit is different for moderately ductile and conventional squat walls. The vertical tension force required to resist overturning can be provided by a combination of concentrated and distributed vertical reinforcement, with the contributions calculated using a plane-sections analysis even though the vertical strain will not vary linearly. The method correctly accounts for compatibility of concrete and reinforcement strains at any point along the wall and satisfies equilibrium.

Recent research by Esfandiari and Adebar (2010) has clarified that only at a height-to-length ratio less than 1.0, does the "fan" mechanism not form and is separate distributed vertical reinforcement required for shear. In these very squat walls, the shear force is resisted by diagonal compression stresses that are relatively uniform across the base of the wall, and the distributed vertical reinforcement for shear is needed to balance the vertical component of this compression.

3.6. Coupled Shear Walls

The use of ductile coupling beams between walls was first suggested in the commentary to the 1973 edition of CSA A23.3. Ductile coupling beam requirements utilizing diagonal reinforcement for high shear forces and short spans, as well as design of the adjacent wall piers were introduced into the 1984 edition of CSA A23.3. *Moderately Ductile* coupled walls are introduced for 2014. This system is essentially a "detuned" version of *Ductile* coupled walls

There are two issues for the design of wall piers: the requirement that the wall bending strength is greater than the coupling beam bending strength, analogous to strong-column weak-beams in frames, and; the requirement that wall piers have adequate vertical tension strength to permit yielding of coupling beams before the formation of a hinge at the base of the walls. Member design forces are typically determined using RSA. Due to higher mode effects, the axial forces at the base are less than the sum of the coupling beam forces over the building height. Thus the initial approach of using a first mode push-over analysis to determine the required vertical tension strength of walls is too conservative and the approach now taken is to scale the vertical strength in proportion to the coupling beam strength. An even more complex issue is how to the increase wall pier strengths in response to increased coupling beam strengths to resist accidental torsion in walls that form a closed-tube. Torsion increases the shear force in coupling beams; but not the axial force in wall piers. Technical committee CSA A23.3 has struggled with this issue and has modified each edition including the 2014 edition; but it remains an issue that needs further research.

4. Design of Gravity-Load Resisting Frames for Seismic Deformations

Perhaps the largest change to the seismic design provisions in CSA A23.3 for 2014 is the requirements for the design of structural members not considered part of the SFRS. One of the most common causes of building collapse during an earthquake is failure of members in the gravity-load resisting frame, e.g., failure of the gravity-load resisting columns. The intent of the requirements is to ensure an adequate level of strength and/or ductility for all structural members subjected to seismically induced deformations. Independent of the ductility of the SFRS, the new requirements must be applied to all buildings unless located in a low seismic region, or the interstorey drift ratio is less than 0.005 at all points in the building.

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 increased displacements of the SFRS due to foundation movements must also be accounted for.

A new interstory drift envelope is given 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 (Dezhdar and Adebar, 2015a). The prescribed envelope also accounts for the nonlinear shear deformation in the plastic hinge region of shear walls. 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. A detailed discussion about the interstory drift envelope given in A23.3-14 and how it can be refined for evaluation of existing buildings is given in Adebar, DeVall, Mutrie and Yathon (2015).

An explanation of the different ways that the interstory drift envelope can be used to determine the demands on the gravity-load resisting frame is given by Adebar, DeVall and Mutrie (2014). When the gravity frame has unique features like a large transfer girder at one level, a computer model of a small portion of the frame can be subjected to displacements that result in the interstory drift for the height of that floor level. It is also possible to subject the entire gravity frame to one displacement profile that results in the interstory drift envelope; however, the resulting axial loads in the gravity columns are larger than the envelope of loads that result from the separate displacement profiles that make up the envelope of 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. Adebar, DeVall and Mutrie (2014) presented solutions for a number of common gravity-load frames.

4.1. Plastic Hinge Regions of Shear Walls

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 is usually 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 (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 (interstory drift changes significantly from one floor to the next), the columns will experience bending because 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. A simple solution to ensuring gravity-load columns will have adequate flexibility is to require that columns have a greater curvature capacity than the curvature demand associated with the inelastic rotational demands on the shear walls and this is the requirement adopted in CSA A23.3-2014.

Over the plastic hinge regions, all columns must contain at least buckling-prevention ties 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 $h_w/200$.

4.2. 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 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.

Recent experimental research (Adebar, 2013) has shown that thin columns and walls have much less toughness than 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_{ro} to reflect their enhanced toughness, thin columns and walls now have a lower $P_{r,max}$ to P_{ro} ratio for gravity load design as shown in Fig. 1. 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 $P_{r,max} = 0.75P_{ro}$. In Canada, spirally reinforced columns now have $P_{r,max} = 0.90P_{ro}$.

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. Walls with a single layer of reinforcement may not be able to tolerate cycles of combined in-plane and out-of-plane displacement. The maximum interstorey drift is used as an indicator of seismic demands, as well as flexibility of the structure.

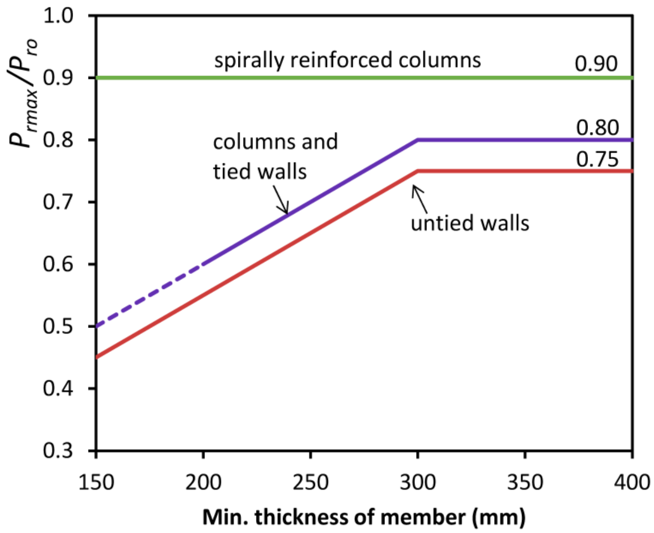

Fig. 1 – Maximum axial load resistance $P_{r\,max}$ for gravity load design of thin columns and walls.

4.3. Flexural Deformation Capacities of Column and Bearing Walls

The new 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 expressed as 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 – resistances are reduced rather than 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 (column, wall or beam), the type of detailing provided in the members (e.g., ductile frame column versus other types of columns), weak or strong axis bending in walls, and the level of applied axial compression. Further information is given in Adebar, DeVall and Mutrie (2014).

5. Seismic Design of Shear Wall Foundations

Another section of CSA A23.3 that has major changes is the requirements for the design of building foundations. Unless a building foundation is restrained from rotating by a structure with sufficient stiffness and strength, the increased displacements due to movements of foundations must be accounted for in the design of the SFRS and in the design of the members not considered part of the SFRS.

If the foundation is capacity protected by the overturning capacity of the SFRS (i.e., SFRS is sufficiently weaker than the foundation), the foundation movements may be calculated using a simple nonlinear static analysis. Adebar (n.d.) presented an equation for estimating the rotations of a shear wall foundation based on the ratio of the required uniform bearing stress to initial shear modulus of the soil and the size of the uniform bearing stress block relative to the size of the footing. This equation has been adopted in CSA A23.3-2014.

When a foundation is not capacity protected (previously called a “rocking footing”), the foundation movements must be estimated using a dynamic analysis that accounts for the reduced rotational stiffness of the footing due to footing uplift and reduced shear modulus of soil. CSA A23.3-2014 includes some upper-bound estimates of the movements that can be used in lieu of doing the complex dynamic analysis. This permits designers to avoid the complex analysis if the structure can easily tolerate the increased building displacements resulting from the reduced strength of the foundation. It is important to note that an undersized shear wall footing will actually protect the shear wall – the inelastic deformation demands will concentrate in the soil below the footing rather than in the shearwall – but will greatly increase the deformation demands on the gravity-load resisting frame.

Based on the results of new nonlinear analysis (Bazargani, Adebar, DeVall and Anderson, 2015), the minimum strength requirements for *not capacity-protected* (NCP) foundations have been changed. The new limits for the minimum overturning capacity of foundations now has two requirements: (a) the factored overturning capacity of the foundation shall not be less than the design forces determined using $R_d\,R_o$ = 2.0, similar to the current NBCC requirement, in addition; (b) the factored overturning capacity of the foundation shall not be less than 75% of the nominal overturning capacity of the SFRS.

When the SFRS has a large overstrength, it may be difficult and unnecessary to capacity protect the foundation or to meet the new requirement for NCP foundations given in (b) above. This leads to the concept of a maximum overturning capacity of a foundation. For typical building structures, the maximum required capacity is needed to resist the design load combinations with earthquake loads calculated using $R_d\,R_o$ equal to 1.3 as typical building structures have a minimum overstrength of 1.3. This approach may be unsafe for foundations, which have overstrength due to the bearing stress in soil or rock being larger than the factored bearing capacity; but an increase in bearing stress may not result in a proportional increase in overturning capacity of the foundation. When the bearing capacity of soil or rock due to seismic overturning is large compared to the uniform bearing stress due to the applied vertical loads, a large increase in the overturning bearing capacity of soil or rock will result in very little increase in overturning capacity of the foundation. Thus the minimum overstrength used to calculate the required maximum overturning capacity of foundations must be applied to the bearing capacity of soil or rock. For 2014, the requirement is that the overturning capacity of a foundation calculated using a bearing stress in the soil or rock equal to 1.5 times the factored bearing capacity need not exceed the overturning moment resulting from design load combinations including earthquake loads calculated using $R_d\,R_o$ equal to 1.0.

The corollary to the above is that when the bearing capacity of the soil or rock is low, overstrength in the bearing capacity may result in a large increase in overturning capacity. That is why the statement in 2010 NBC that “the design forces for the SFRS need not exceed the maximum values associated with foundation rocking” is unsafe. The overturning capacity of a “rocking” foundation may be much larger than estimated and thus the forces applied to the SFRS may be much larger. The statement has been removed in 2015 NBC.

When a foundation is capacity protected by the SFRS, the forces applied to the footing are well defined. Similarly, when a foundation is designed for the maximum required capacity, the foundation design forces can safely be used to design the footing. When a foundation is not capacity protected, special considerations are needed to determine the design forces for the footing. The plan dimensions of the footing, which control the foundation overturning capacity, are determined using the factored bearing capacity of the soil or rock. Overstrength in bearing capacity of soil or rock results in the same resultant vertical force; but that force is located closer to the “toe” of the foundation. Thus the required amount of flexural reinforcement in the footing must be increased, and the shear design of the footing may need to account for the increased shear span-to-depth ratio of the footing.

A summary of all the new foundation design requirements and the background research that lead to these requirements are presented by Adebar, DeVall, Bazargani and Anderson (2014).

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