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ACCOUNTING FOR HIGHER MODE SHEAR FORCES IN CONCRETE WALL BUILDINGS: 2014 CSA A23.3

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ABSTRACT: The traditional approach is to reduce elastic shear force demands using the same force reduction factor used to reduce bending moment demands accounting for flexural ductility. The 2004 edition of CSA A23.3 required that the magnification of shear due to inelastic effects of higher modes be accounted for; but gave no guidance and designers ignored the requirement. The commentary stated that no suitable simple method was available; but it is expected that a solution will soon be available. This paper summarizes the research that led to the implementation of a simple procedure to account for the magnification of shear due to inelastic effects of higher modes in the 2014 edition of CSA A23.3. While US designers of high-rise shear wall buildings that use nonlinear response history analysis (NLRHA) typically use an amplification factor of 3.0, the amplification factor in CSA A23.3-2014 is limited to 1.5. This was done because concrete shear walls have consider shear ductility, the maximum shear force demand only occurs in one short cycle and the shear demand in all other cycles is considerably less, and the maximum shear force does not occur simultaneously with the maximum base rotation, while the calculation of shear resistance assumes that it does.

1. Introduction

One of the greatest challenges in the design of high-rise concrete shear wall buildings is accounting for the shear force demands due to higher modes. The traditional design approach is to reduce the shear force demands determined from a linear dynamic (response spectrum) analysis (RSA) using the same force reduction factor that is used to reduce the bending moment demands accounting for flexural ductility. It has been known for a long time that the shear force demands do not reduce proportionally to the maximum bending moment at the base of taller buildings because the maximum bending moment is primarily due to the first mode while the shear force has large contributions from the higher modes.

Designers of high-rise shear wall buildings in the US that use nonlinear dynamic analysis (performance-based design) are designing for shear forces that are 2 to 3 times larger than the shear forces determined using the traditional approach. Many high-rise buildings in both Canada and the US continue to be designed for the lower shear force demands using the traditional approach.

The 2014 edition of CSA A23.3 contains new design requirements for the shear force demands in high-rise concrete shear wall buildings accounting for higher modes. The increase in shear force from the traditional approach, i.e., the shear force reduced proportional to bending moment, is limited to 1.5, which is about half the increase described above. This paper presents the background to the new requirements and the justification for the lower increase in shear force demand.

2. Some Important Background Literature

Considerable research has previously been done and continues to be done on this topic. The following is a brief summary of some important background literature. The first work to recognize the shear amplification problem was Blakely et al (1975), who demonstrated through nonlinear analyses that shear demands in cantilever shear walls were larger than calculated by a code static analysis. They observed that the amplification increased with the height of the structure and with the level of seismic input. Using their work, the following equations were included in the 1982 New Zealand Design Standard (NZS, 1982), and was included as reference information in the commentary to the 1994 version of CSA A23.3 (CSA, 1994):

$$\omega_s = 0.9 + \frac{N}{10}\ for\ N < 6 \qquad (1)$$

$$\omega_s = 1.3 + \frac{N}{30} \leq 1.8\ for\ N \geq 6$$

where ω_s is the amplification from a static analysis, and N is the number of storeys. Note that there is an amplification for all story heights above one (N = 1) and the amplification is constant above 15 stories (N = 15).

Many modern approaches attempt to represent the shear amplification in terms of the linear elastic modes. Eibl and Keinztel (1988) and Keintzel (1992) proposed a model where yielding due to the forces in the first mode reduced the shear force in that mode only, based on observations that the shape of the other modes and their forces remained similar to the elastic modes after base yielding. Using this concept, Eurocode 8 adopted the following equation:

$$\omega_v = q\gamma\sqrt{\left(\frac{M_y}{qM_I}\right)^2 + 0.1\left(\frac{S_{a,max}}{S_a(T_1)}\right)^2} \leq q \qquad (2)$$

where ω_v is the amplification factor to be applied to a response spectrum analysis (RSA), M_y is the base yield moment, q is the ductility factor, M_I is the base moment from an RSA, $S_{a,max}$ is the maximum spectral acceleration, $S_a(T_1)$ is the spectral acceleration of the first period, and γ is a correction factor normally equal to 1.0.

Despite the theoretical background, the equation by Eibl and Keintzel (1988) does not explicitly consider the forces from the modes. Priestly and Amaris (2003) attempted to use the forces from the modal analysis directly:

$$V_a = \sqrt{V_{y1}^2 + V_{e2}^2 + V_{e3}^2 + \cdots} \qquad (3)$$

where V_a is the amplified shear, V_{ei} is the elastic shear from the i^{th} mode, and V_{y1} is the shear associated with yielding in a first mode distribution. Other work by Sullivan et al (2006) has attempted to use the modes modified by yielding directly in the formulation of their shear amplification models.

Recently, Pugh (2012) has proposed a modification to the modal method. While most methods assume that only the first mode forces cause the yielding and should be reduced, Pugh (2012) showed that this assumption overestimates the amplification for taller structures, and proposed a method where only the shear from the mode that contributes the most to the demand is reduced by the ductility factor. He showed that this method agreed well with results from an extensive series of nonlinear analyses.

Most of the methods proposed for calculating the shear force demands in high-rise shear wall buildings include three main parameters: the height of the building (or the first-mode period), the ductility force-reduction factor used to reduce the bending moment and shear force determined from a linear analysis, and the shape of the design spectrum. Rutenberg (2013) recently presented a comprehensive summary of other models that have been proposed.

Most of the methods described above appear very similar; but do produce very different results. Figure 1 compares the results from these different methods using the Vancouver design spectrum in the 2010 NBCC (NRC 2010). The results are expressed in terms of an amplification factor, which is the ratio of the correct design shear force at the base of the wall to the traditionally calculated shear force. The latter is equal to the shear determined from a linear dynamic analysis divided by the same flexural ductility factor used to reduce the bending moment. Figure 1 was developed assuming a flexural cantilever of uniform mass and stiffness, with a fundamental period equal to the number of storeys divided by ten.

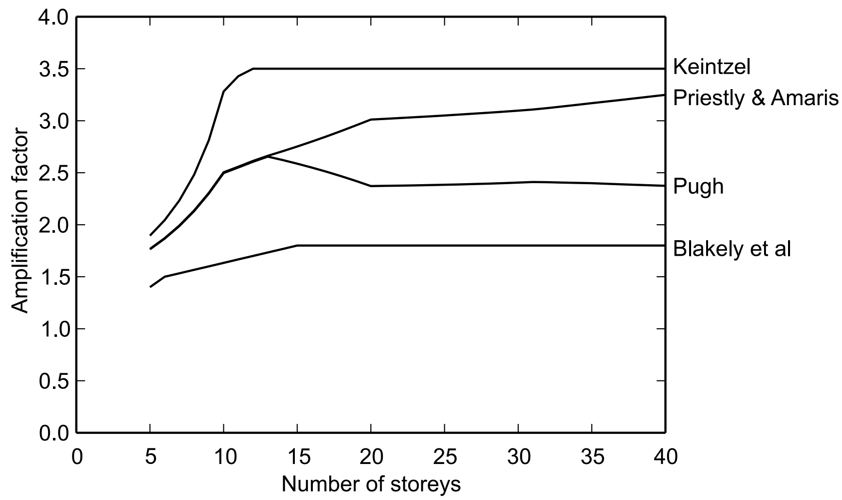

Fig. 1 – Comparison of results from four different methods for determining shear amplification factor for case where elastic shear force demand has been reduced using a ductility force-reduction factor of 3.5.

3. Mechanics of the Problem

The shear amplification phenomenon is typically thought to be the result of inelastic action of higher modes. A physical explanation for these inelastic modes is described briefly in this section. The main issue is the shear force demand being reduced from the elastic demand by the same force reduction factor as the bending moment. This is common practice, despite it being well known that the contributions to the shear and the moment between modes do not vary in the same ratio. Once the moment causing yielding is established, it is simple to come up with a realistic combination of modes that causes shear forces larger than the design shear force. This “elastic shear amplification” will be observed later from the nonlinear time history results. Both elastic and inelastic amplification are present.

As described initially in Keinztel (1992), the “inelastic higher modes” can be thought of as the elastic modes when a pin is placed at the base of the cantilever wall. The first mode becomes a mechanism, but the higher modes still cause shear forces over the height of the structure. This is shown in Figure 2.

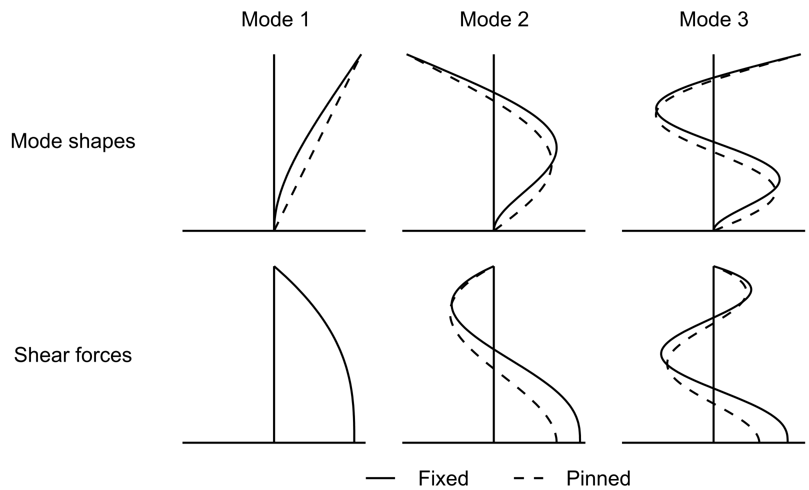

Fig. 2 – Mode shapes and shear force envelopes for fixed-based and pinned-based cantilevers.

4. Nonlinear Response History Analysis

Nonlinear response history analysis (NLRHA) was used to study the shear demand in thirteen different cantilever shear wall buildings. The main parameters were height of building and strength of shear walls, which may be defined by the parameter R_g = the ratio of elastic bending moment demand at the base of the walls calculated using the uncracked flexural stiffness EI_g to the nominal flexural strength M_n. The study included three 10-story buildings with R_g = 1.7, 2.6, and 4.2; one 20-story building with R_g = 4.0; four 30-story buildings with R_g = 1.4, 2.4, 3.1, and 4.3; one 40-story wall with R_g = 4.4; and four 50-story walls with R_g = 1.4, 2.1, 2.4, and 4.1. The thirteen buildings were designed according to the requirements of CSA A23.3-04. The amount of vertical reinforcement at the zones was determined considering the level of axial compression force due to gravity loads and the target R_g. The full description of the shear walls can be found in Dezhdar (2012).

The input ground motions in this study were scaled to the uniform hazard spectrum (UHS) for Vancouver BC for site class C. 80 ground motions with moment magnitude between 6.5 and 8.0 and closet source to site distance between 0.5 and 50 km were included. A comprehensive study was carried out to investigate the influence of ground motion selection and scaling on various demand parameters (Dezhdar and Adebar, 2015). Time history results presented in this study correspond to the ground motions that were matched to the target UHS over a range of periods wider than $0.2T_1$ to 1.5 T_1, where T_1 is equal to 1.0, 2.0, 3.0, 4.0, and 5.0 seconds for 10, 20, 30, 40, and 50 story shear walls, respectively.

The nonlinear response history analysis of the 13 shear walls was done using OpenSees (2008) with a specially developed trilinear hysteretic bending moment - curvature relationship. Details of the hysteretic model can be found in Dezhdar (2012). The parameters that define the trilinear model were calculated at each floor considering the variation of axial compression. A force element was defined at each floor level to model the vertical spread of plasticity in the walls. A fixed base was assumed, and shear deformations were not considered in the model. Rayleigh damping was assumed with mass proportional and initial stiffness matrixes. A damping ratio of 3% was assigned for the first and third modes.

Figure 3 summarizes the results of the NLRHA in terms of a shear amplification factor, which is the amount the design shear force (reduced from an elastic analysis) needs to be increased again. An effective stiffness of $0.5EI_g$ was used to compute the elastic bending moment and shear force demands. To obtain design shear forces, elastic shear forces were reduced by the ratio of elastic bending moment at the base corresponding to $0.5EI_g$ to wall flexural strength M_n.

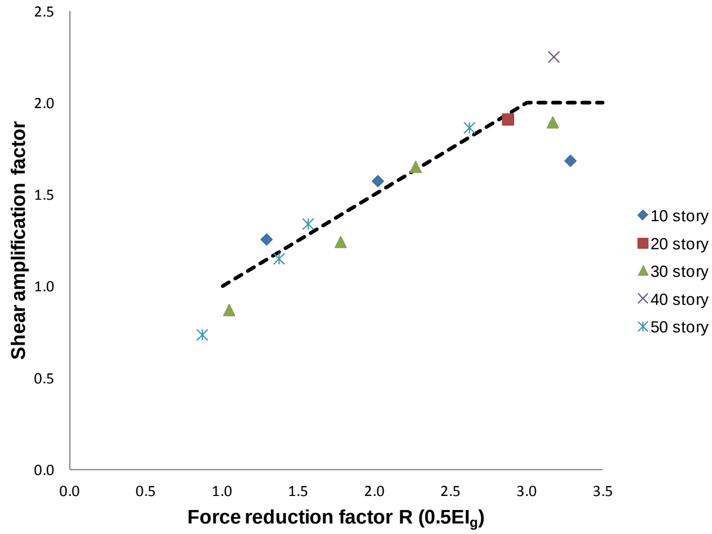

Fig. 3 – Shear amplification factors determined from NLRHA of thirteen shear wall buildings.

5. Reasons for Lower-bound Shear Amplification Factor

5.1. Shear Cracking and Shear Ductility

Most nonlinear response history analysis (NLRHA) uses a simple linear model for the shear response. This very primitive model does not capture some of the important nonlinear shear behaviour.

Rad and Adebar (2008) showed that reduced shear stiffness due to diagonal cracking reduced the maximum shear force at the base of the wall. For R = 5, a reduction in shear rigidity to 20%, 10% and 5% of $G_c A_v$ reduced the maximum shear force at the base to 87%, 73% and 55% of the shear force determined using the uncracked section shear rigidity.

Using a nonlinear shear model (Gerin and Adebar, 2009), Rad (2009) investigated whether providing a lower shear capacity than indicated by linear analysis would result in a shear failure. The results from one example are shown in Fig. 4 for a cantilever wall designed using a force-reduction factor of 3.5 for both the bending moments and the shear forces. The maximum shear strains observed in the wall for different ground motions are shown as the ordinate in Fig. 4. When the shear strength factor was 1.0, i.e., a shear amplification factor of 1.0 used to design the wall, the maximum shear strains exceeded the acceptable limit of 0.007 (shown as solid line in Fig. 4); however when the wall was designed using a shear strength (amplification) factor of 1.5, net shear force reduction factor of 3.5/1.5 = 2.3, all shear strains were within the acceptable limit. This demonstrates that the wall does not have to be designed for the elastic shear demands as the wall can tolerate some yielding of horizontal reinforcement in the wall.

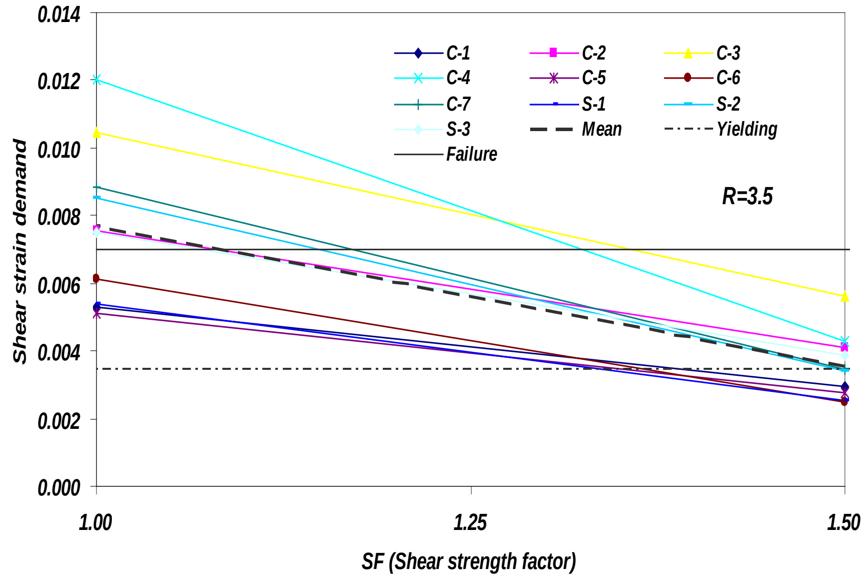

Fig. 4 – Influence of shear strength (amplification) factor used to design the horizontal reinforcement on the maximum shear strains; from Rad (2009).

5.2. Maximum Shear Force Demands in Cycles

When NLRHA is used to determine the shear force demand, the single largest shear force peak that occurs during one load cycle is used. It is important to note that all other peak shear force demands will be smaller. The question is how much smaller.

The peak shear force demands from all cycles were determined for 10, 30, and 50 story shear wall buildings with force reduction factors of 3.2, 3.1, and 3.7, respectively. For each ground motion, peak base shear force at various cycles was normalized to the maximum base shear force over the entire ground motion, and a curve was determined that related the normalized base shear force to the number of cycles. For a given number of cycles, the mean value for a suite of ground motions shows how much the base shear force drops compared to the maximum value.

Figure 5 shows the mean plots (from many ground motions) for the three buildings. Of course the cycles generally do not occur in the order of the shear force demand, and Fig. 5 merely shows the mean ratio of maximum base shear force at the n^{th} biggest cycle to the maximum base shear force. Figure 5 indicates that after 5 cycles, the base shear force drops to 80% of the maximum value for the 10 and 30-story buildings and to 73% of the maximum value for the 50-story building (see dashed line). If there is no shear failure in the cycles with maximum base shear force, the mean base shear demand reduces by 20% or 27% after only 5 cycles and continues to reduce further for additional cycles.

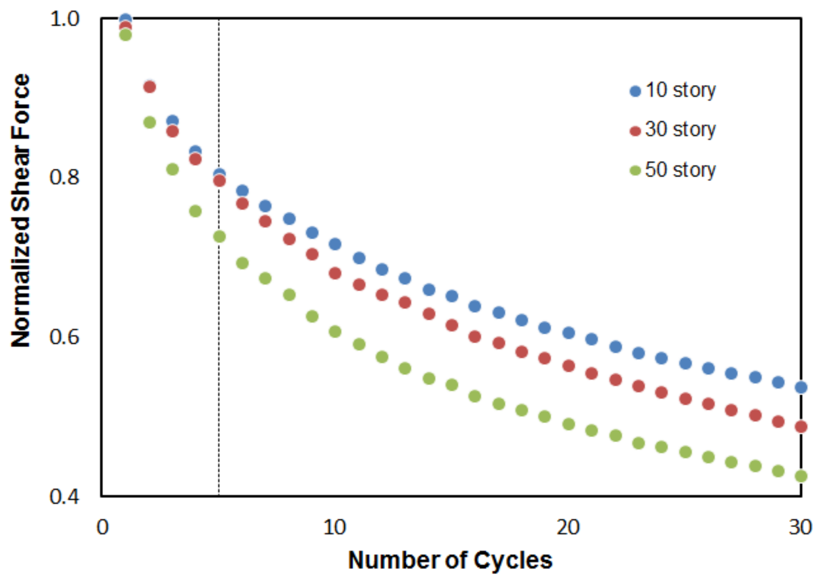

Fig. 5 – Reduction in shear force demands in other cycles.

5.3. Shear – Wall Rotation Interaction

The final issue that justifies using a lower-bound shear force demand is that the CSA A23.3 requirements for seismic shear (Adebar, 2006) assume the maximum wall rotation and curvature demands associated with the maximum displacement demands will occur at the same time as the maximum shear force demand. The results from the NLRHA (Dezhdar, 2012) was used to investigate this issue.

Figure 6 below compares the interaction of wall curvatures and shear force. The results are shown for individual ground motions, as well as the mean result and the mean plus one standard deviation result. The plots in Fig. 6 show that there is a strong interaction – the maximum shear forces and maximum curvatures do not occur at the same time.

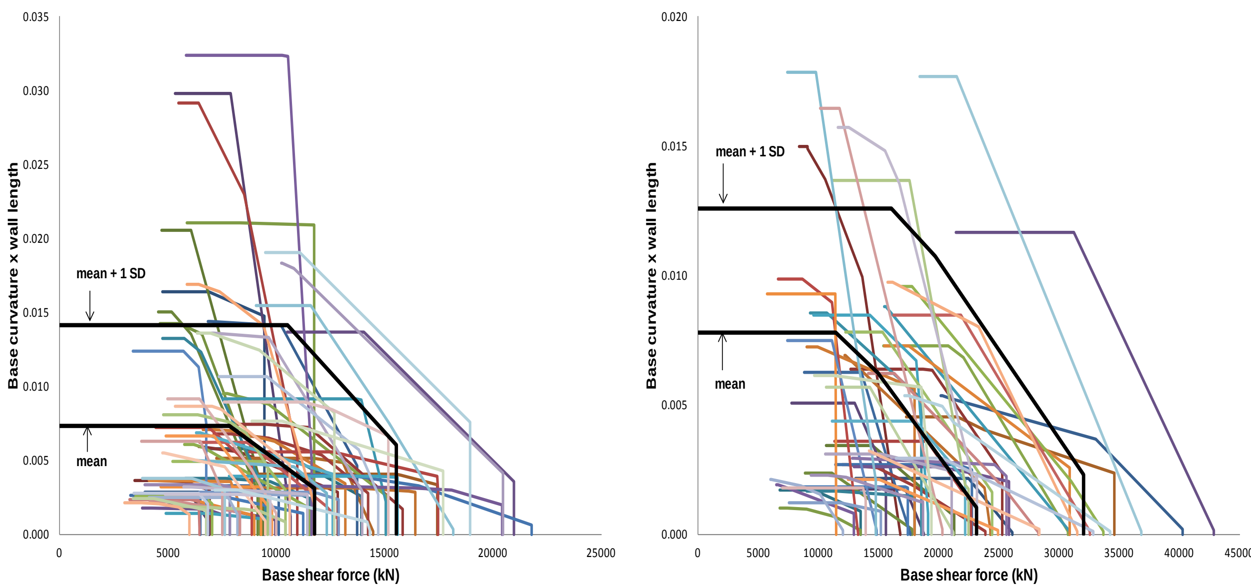

Fig. 6 – Interaction between base curvature and base shear force in: 10-story shear wall building (left); 30-story shear wall building (right).

6. 2014 CSA A23.3 Requirements

The 2004 edition of CSA A23.3 required that shear walls have a factored shear resistance greater than the shear due to the effects of factored loads accounting for the magnification of the shear due to the inelastic effects of higher modes (Clause 21.6.9.1); however, no guidance was provided on how this requirement can be satisfied. The commentary to Clause 21.6.9.1 stated: "*unfortunately, no simplified method is available at this time to account for the magnification of shear due to inelastic effects of higher modes by simply modifying the results of a linear analysis. It is hoped (expected) that such a simplified solution will be available soon.*" Most designers ignored this requirement because they did not know how to satisfy it.

Based on the results presented in this paper, the 2014 edition of CSA A23.3 contains the following new clause:

21.5.2.2.7 Accounting for inelastic effects of higher modes

Except for coupled and partially coupled shear walls, the factored shear force, increased to account for flexural overstrength, shall be further increased depending on the fundamental lateral period of vibration of the building T_a in the direction under consideration as follows:

$T_a \le T_L$	$T_a \ge T_U$
1.0	$\mathbf{1.0 + 0.25\left({R_d R_o}/{\gamma_w} - 1\right) \le 1.5\ ;\ \ge 1.0}$

For T_a between T_L and T_U, linear interpolation shall be used. For western Canada, T_L = 0.5 s and T_U = 1.0 s.

R_d and R_o are the ductility-based and overstrength-based force reduction factors specified in the National Building Code of Canada. γ_w is the wall overstrength factor equal to the ratio of the load corresponding to nominal moment resistance of the wall system to the factored load on the wall system, but need not be taken as less than 1.3. The term ($R_d\ R_o\ /\ \gamma_w$) is analogous to the force reduction factor R given in Fig. 3.

The amplification factor given in the table above reaches the maximum value when $R_d\ R_o\ /\ \gamma_w = R = 3.0$, which is consistent with the results shown in Fig. 3; however, the maximum amplification factor is limited to 1.5 instead of 2.0 as indicated in Fig. 3. The reasons why the CSA technical committee decided to do this was because of the three reasons described in Section 5 above: (i) concrete shear walls have consider shear ductility, (ii) the maximum shear force demand determined in a NLRHA only occurs in one short cycle and the shear demand in all other cycles is considerably less than the maximum value, and (iii) the maximum shear force does not occur simultaneously with the maximum base curvature (rotation), while the calculation of shear resistance in CSA A23.3 assumes that it does.

7. References

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