DYNAMIC SHEAR AMPLIFICATION IN HIGH-RISE CONCRETE WALLS: EFFECT OF MULTIPLE FLEXURAL HINGES AND SHEAR CRACKING

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ABSTRACT :

Concrete walls are becoming a very popular seismic force resisting system for high-rise buildings up to 600 ft (180 m) high along the west coast of North America. One critical design issue is what shear strength is required to ensure a brittle shear failure will not occur. Linear dynamic response spectrum analysis (RSA) is usually used in Canada to determine the relationship between maximum bending moment and maximum shear force. Nonlinear time history analysis (NLTHA) has shown that flexural yielding of the wall does not limit the shear force in the wall, and that scaling the maximum bending moment and maximum shear force from a response spectrum analysis by the same reduction factor may result in unsafe design. The shear force tends to increase as the magnitude of ground motion is increased, and this is referred to as dynamic shear amplification.

The influence of flexural yielding at multiple locations over the wall height and influence of shear deformations due to diagonal cracking of the wall were investigated. The results indicate that both significantly reduce the maximum shear force in the wall. When multiple hinges occurred and the ratio of maximum bending moment at base determined from RSA to flexural strength at base was equal to 5, the maximum shear force near the top of the wall reduced 50%, while the maximum shear force at the wall base reduced 20%. Reduced shear stiffness due to diagonal cracking further reduced the maximum shear force at the base of the wall. When the shear rigidity of a cracked concrete wall is equal to 10% of the uncracked section shear rigidity, which is a typical value, the maximum shear force at the base reduced a further 27%. The combined influence of multiple hinging and diagonal cracking reduced the maximum seismic shear force at the base of the wall by 40%.

KEYWORDS: Reinforced concrete, walls, seismic shear, nonlinear analysis, design forces.

1. INTRODUCTION

Many high rise concrete wall buildings are designed in North America using only linear dynamic response spectrum analysis (RSA) to determine the seismic forces acting on the walls such as the bending moment and shear force envelopes. These buildings are designed using ductility force reduction factors *R* up to about 5. Thus the maximum bending moment at the base of the wall determined by RSA is reduced by up to a factor of 5 because the wall has adequate ductility, which means the displacement capacity of the wall after a plastic hinge forms at the base is greater than the displacement demand. The design shear force at the base of the wall has traditionally been reduced from the elastic shear force determined from RSA by the same force reduction factor used to determine the design bending moment.

Nonlinear dynamic analysis has shown that flexural yielding of a cantilever wall does not limit the shear force in the wall. The shear force tends to increase as the magnitude of ground motion is increased. This increase in shear force is often referred to as "dynamic shear amplification." The dynamic shear amplification factor is the ratio of shear force demand obtained from nonlinear analysis to shear demand obtained from a simplified code static procedure or linear dynamic RSA. The amplification, which is attributed to the influence of higher modes on a cantilever wall with a hinge at the base, can be as large as 3 or even more.

2. PREVIOUS WORK

Various empirical approaches have been proposed to estimate the design seismic shear force using a dynamic shear amplification approach where the shear force is first reduced using the force reduction factor due to flexural ductility and then increased again to account for the influence of higher modes on a cantilever wall with a plastic hinge at the base. Blakeley, Cooney and Megget (1975) were among the first to propose a simple dynamic shear amplification factor, which appeared in the 1982 New Zealand Design Standard. Their amplification factor ω_v is a piece wise linear function of the number of stories (n) and is described by the following points: $\omega_v = 1$ at $n = 1$, $\omega_v = 1.5$ at $n = 6$, and $\omega_v = 1.8$ at $n \geq 15$.

Keintzel et al. (1992) proposed a simple formula to estimate the dynamic shear amplification factor assuming the ductility force reduction factor should only be used to reduce the shear force associated with the fundamental mode, and the amplification due to higher modes depends on the ratio of maximum design acceleration at any period to design acceleration for the fundamental period. Using a similar approach, Ghosh (1992) proposed the maximum shear demand at the base of isolated cantilever walls is equal to the sum of the shear force associated with the bending moment capacity of the wall with the resultant force acting at two-thirds the height of the wall (as in the first mode), plus 25% of the inertia force due to the peak ground acceleration acting on the total weight of the building. Eberhard and Sozen (1993) proposed a similar expression except the 25% factor ranged from 27 to 30%.

Seneviratna and Krawinkler (1994) and more recently, Rutenberg and Nsieri (2005), studied the seismic shear force distribution when flexural hinge forms at the base of a wall. The latter proposed a simple seismic shear envelope that depends on the fundamental period of the structure. The minimum design shear force over the height of the wall is 50% of the maximum at the base.

3. CURRENT STUDY

In all of the previous studies on dynamic shear amplification, the response of the concrete wall was assumed to be linear above the flexural plastic hinge region at the base of the wall and the influence of reduced shear stiffness due to diagonal cracking was not accounted for. In the current study the influence of flexural yielding over the height of the wall and influence of shear deformations due to diagonal cracking of the wall were investigated.

The nonlinear dynamic analysis was done using a single cantilever wall to represent the entire core of a typical high-rise building in Canada. The wall had a total height of 81.0 m and a story height of 2.7 m. The mass concentrated at each story (uniform over the height of the building) was adjusted to give a fundamental period of 3.0 s, which is a typical value for a building of this height. The cross-section of the wall representing the entire core had an I-shape with a "web" length equal to 9.0 m, a "flange" width of 9.0 m and a uniform thickness of the "web" and "flange" of 0.75 m. The concrete compressive strength was assumed to be $f_c' = 60$ MPa. The wall geometry was assumed to be constant over the height of the wall. The wall was subjected to a linearly varying axial compression from zero at the top to 10% $f_c' A_g$ at the base. P-Delta effect was included in the analyses. The Takeda model was used to simulate the hysteretic bending behaviour of the wall, and the time step used in the analysis was 0.001 s. SAP 2000 was used to do all analyses.

In order to perform nonlinear time history analysis, ten ground motion records scaled to the 2005 NBCC design spectrum for Vancouver were chosen. The records were scaled by multiplying all acceleration values in one record by the same scale factor. In the case of high-rise buildings, the influence of higher modes of vibration is significant and therefore a wide range of periods need to be considered for scaling. ATC 7-05 suggests a period range between $0.2T_1$ and $1.5T_1$ in which T_1 is the fundamental period of vibration. A period range of 0.5 to 4.5 s was used to determine the appropriate scaling factor as the wall had a fundamental period of T_1=3.0 s and a second mode period of T_2=0.57 s.

30 ground motion records (20 crustal ground motions for site class C from ATC 40, and 10 subduction motion recorded at the Tokachi-Oki event in Japan) were scaled and the best fit seven crustal motion records (C1 to C7) and best fit three subduction motion records (S1 to S3) were selected. The ground motions were selected based on the best fit obtained over the period range 0.5 to 4.5 s. The spectrums for the 10 selected, which were scaled using factors between 0.8 and 3.6, are shown in Fig. 2.

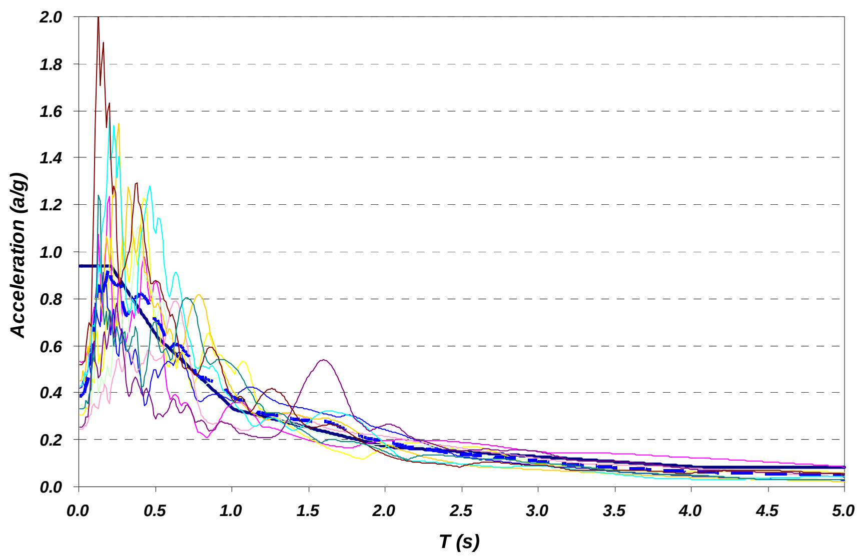

Fig. 1 – Spectrums of 10 scaled earthquakes (thin solid lines), the average spectrum (thick dashed line) and the 2005 NBCC design spectrum for Vancouver (thick solid line).

4. RESULTS FROM ANALYSES

The 10 ground motion records were used to do linear time history analyses (LTHA) of the core wall. The bending moment envelopes and shear force envelopes determined from LTHA were compared with the envelopes determined from RSA. In general, the average of the 10 LTHA envelopes was similar to RSA envelope, particularly the magnitudes of the maximum bending moment and maximum shear force at the base of the wall. The largest difference was in the shape of the bending moment envelopes. The RSA gave a bending moment envelope that was almost linear, while the LTHA gave bending moment envelopes that were about 20% larger at mid-height and about 20% smaller at the quarter height. The shear force envelopes were in general more similar, and again the LTHA gives envelopes that are more nonlinear over the height than the RSA envelope.

Influence of Flexural Yielding Over Height

Nonlinear time history analyses (NLTHA) were performed for three different levels of wall bending strength at the base. The strength was quantified in terms of the ratio R equal to the maximum bending moment at the base determined from RSA to the flexural strength at the base. Analyses were done for R = 2, 3.5 and 5.

The nonlinear time history analysis (NLTHA) of the core wall was done in two different ways. In the first case the wall was allowed to yield only in the plastic hinge zone defined at the base. This is similar to what was done in all previous studies. To ensure sufficient length of plastic hinging, nonlinear elements were used over a height of 18.9 m, which is equal to 7 stories. The rest of wall above the plastic hinge was forced to stay linear. The bending moment envelopes determined from the 10 NLTHA for R = 5 are shown in Fig. 2, while the corresponding shear force envelopes are shown in Fig. 4. The average of the 10 envelopes is also shown in each figure.

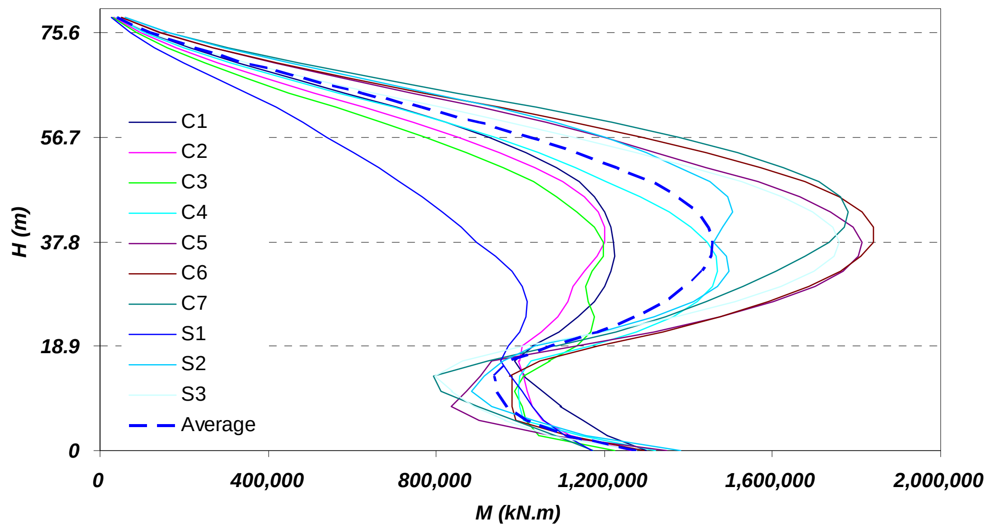

Fig. 2 – Bending moment envelopes determined from analysis with plastic hinging permitted only at the base of wall (R =5).

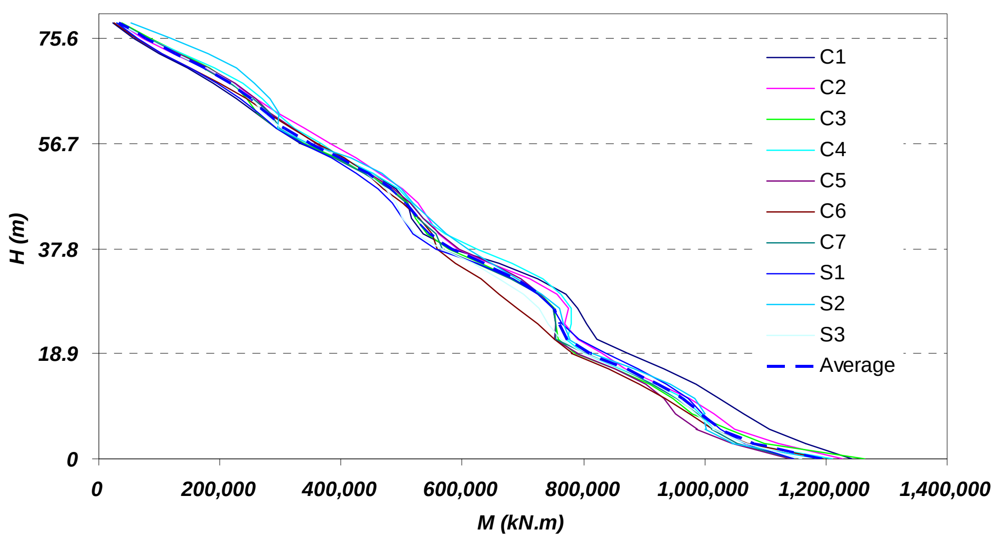

Fig. 3 – Bending moment envelopes determined from analysis where flexural hinging permitted at any location over height of wall (R =5).

Fig. 2 indicates that the bending moments at the mid-height of the wall are greater than the bending moment at the base of the wall. As the axial compression in the wall will be about half the axial compression at the base, and the axial compression resists a significant part of the bending moment on a core wall, it is certain that the wall will yield in flexure near the mid-height.

The second type of NLTHA that was done was to allow plastic hinging to occur at any location over the height of the wall. That is, nonlinear flexural elements were provided over the full height. As described earlier, RSA gives a bending moment envelope that varies approximately linearly over the height. The axial compression in the wall, which often resists about 50% of the bending moment, also varies linearly over the height. Thus the vertical reinforcement in the wall needs to reduce linearly over the height. In reality, the amount of reinforcement is reduced in steps over the height of the structure. The number of times the vertical reinforcement is reduced over the height varies with the design engineer. For simplicity, the variation of the bending strength of the wall was assumed to have four steps over the height. The magnitudes of the four different strengths were chosen to fit the RSA bending moment envelope as would be done in design.

The bending moment envelopes determined from the 10 NLTHA with flexural hinging over the height and with R = 5 are shown in Fig. 3, while the corresponding shear force envelopes are shown in Fig. 5. The average of the 10 envelopes is also shown in each figure. The bending moment demand envelopes shown in Fig. 3 indicate that the wall yields at four places over the height, thus the bending moment demand envelopes are very similar to the bending moment strength envelope over the height. It is important to note that Figures 2 and 3 are drawn at very different scales.

A comparison of the shear force envelopes in Figures 4 and 5 indicate that the flexural yielding over the height has a significant influence on the corresponding shear force envelopes. The maximum shear force at the base of the wall reduced by 20% due to the multiple flexural hinges over the height. Even more significant is the elimination of the higher-mode "bulge" in the shear force envelope near the top of the wall where the shear force reduced by 50%. The influence of multiple flexural hinges can be expressed in terms of the change to the dynamic shear amplification. At the wall base, multiple hinging reduced the dynamic shear amplification from 1.5 to 1.3 (R = 2), from 2.3 to 2.0 (R = 3.5), and 3.1 to 2.5 (R = 5). Near the top of the wall, the multiple flexural hinges reduced dynamic shear amplification from 1.5 to 1.1 (R = 2), 2.4 to 1.4 (R = 3.5) and 3.2 to 1.6 (R = 5).

Influence of reduced shear stiffness due to diagonal cracking

None of the previous studies have accounted for the influence of shear cracking or shear yielding on the dynamic shear magnification. A simple way to consider the influence of diagonal cracking is to use a reduced effective shear rigidity of the wall. Prior to cracking, the effective shear rigidity G_cA_{ve} can be taken equal to the uncracked section shear rigidity $G_c A_{vg}$. After cracking, the shear rigidity reduces depending on the amount of cracking. Gérin and Adebar (2004) have found that a typical effective shear rigidity of diagonally cracked concrete walls is 10% $G_c A_{vg}$.

The NLTHA with multiple flexural hinges over the height was repeated using a range of effective shear rigidities $G_c A_{ve}$ equal to 20%, 10% and 5% of $G_c A_{vg}$. The previous analyses were done using a shear rigidity equal to $G_c A_{vg}$. The analyses were again done for three levels of wall flexural strengths corresponding to R equals 2, 3.5 and 5. The results for R = 5 are summarized in Fig. 6. The average shear force envelope shown in Fig. 5 is repeated in Fig. 6 and is labeled G_cA_{ve} = 1.0 $G_c A_{vg}$. Also shown in Fig. 6 are the three other average shear force envelopes. That is, each line in Fig. 6 represents the average shear force envelope determined from 10 NLTHA.

The reduction in shear rigidity of the wall has a very significant influence on the maximum seismic shear force at the base of the wall. The reduction in shear rigidity to 20%, 10% and 5% of $G_c A_{vg}$ reduced the maximum shear force at the base to 87%, 73% and 55% of the maximum shear force determined using the uncracked section shear rigidity. The influence of reduced shear rigidity can also be expressed in terms of the change to the dynamic shear amplification. Table 1 summarizes the dynamic shear amplification at the base of the wall for all the different NLTHA that were done.

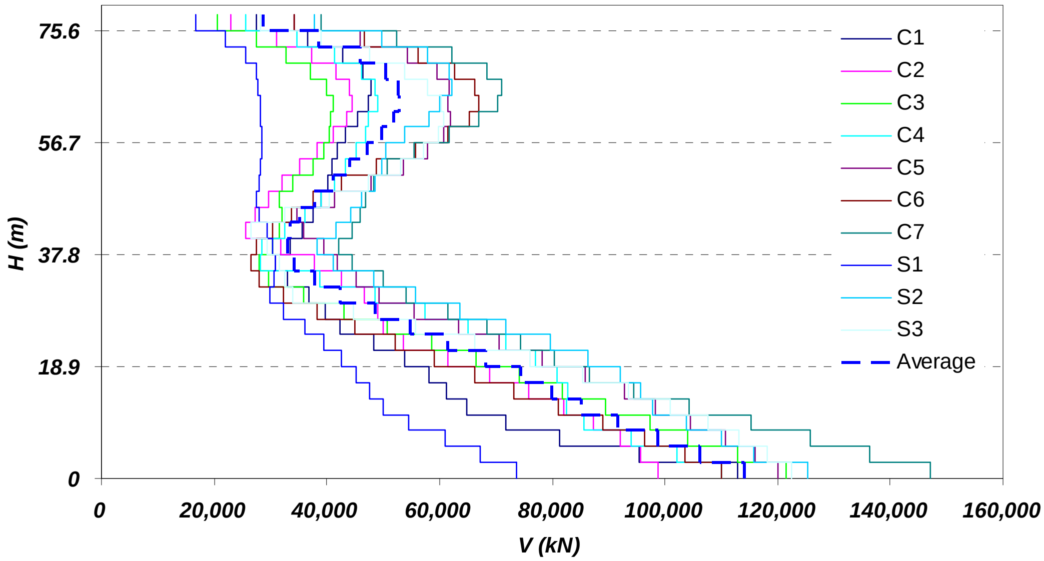

Fig. 4 – Shear force envelopes determined from analysis with plastic hinging permitted only at the base of wall (*R* =5).

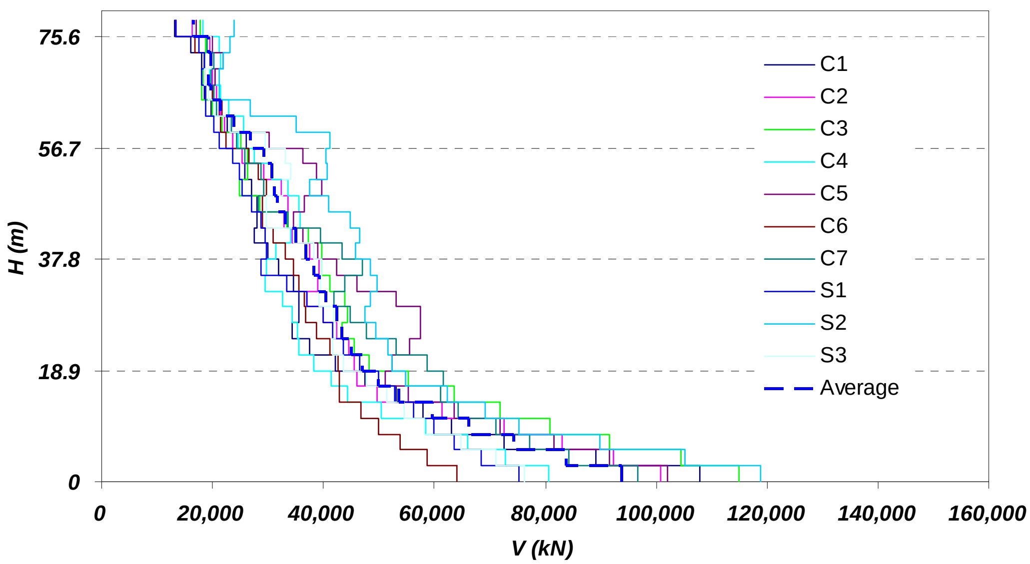

Fig. 5 – Shear force envelopes determined from analysis where flexural hinging occurs at many locations over height of wall (*R* =5).

5. CONCLUDING REMARKS

The results of the current study have shown that flexural yielding of the wall at numerous locations over the height reduces the maximum shear force in the wall. When the ratio *R* of maximum bending moment at base determined from RSA to flexural strength at base was equal to 5, the maximum shear force near the top of the wall reduced 50%, while the maximum shear force at the base of the wall reduced to 80% of the value determined from an analysis where hinging occurs only at the base. The results of the current study have also shown that reduced shear stiffness due to diagonal cracking further reduces the maximum shear force at the base of the wall. For *R* = 5, a reduction in shear rigidity to 20%, 10% and 5% of $G_c A_{vg}$ reduced the maximum shear force at the base to 87%, 73% and 55% of the shear force determined using the uncracked section shear rigidity. The combined influence of multiple hinging over the height of the wall and reduced shear rigidity due to diagonal cracking reduced the maximum seismic shear force at the base of the wall to 71%, 60% and 45% of the shear forced determined from a NLTHA where flexural yielding is limited to the base and the uncracked shear rigidity is used for the wall. Thus it is very important to account for these two phenomenon whenever conducting a NLTHA to determine the maximum seismic shear force in a concrete wall.

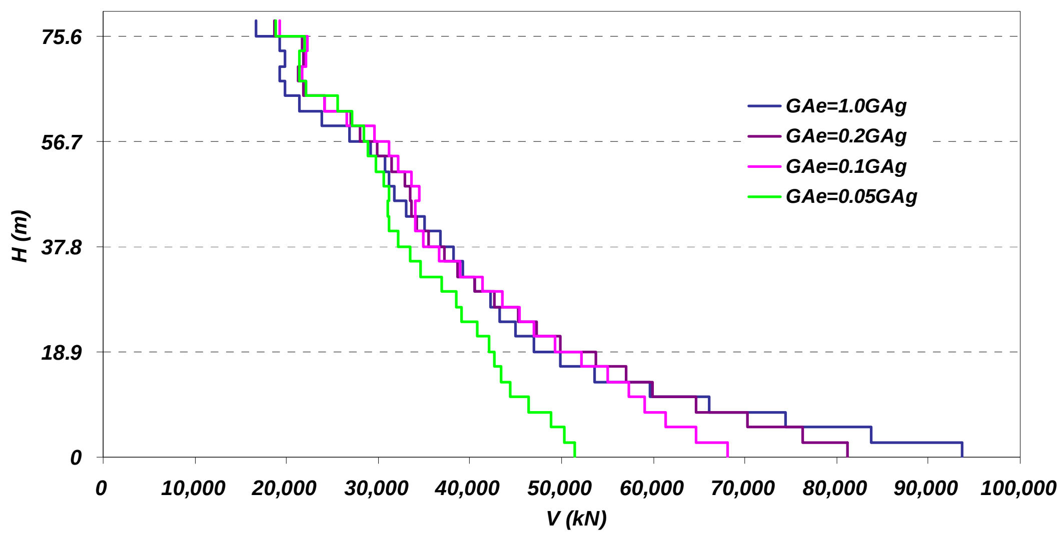

Fig. 6 – Influence of effective shear stiffness on average shear force envelopes determined from analyses where flexural hinging occurs at many locations over height of wall. Each average envelope is from 10 NLTHA with R = 5.

Table 1 – Average dynamic shear amplification at base of wall determined from comparing results of 10 NLTHA with results of RSA.

R	**Flexural Yielding**	**Shear Stiffness GA_{ve}/GA_{vg}**	**Average Dynamic Shear Amplification**
2.0	Single hinge at base	1.0	1.48
	Multiple hinges	1.0	1.32
		0.2	1.06
		0.1	0.94
		0.05	0.79
3.5	Single hinge at base	1.0	2.34
	Multiple hinges	1.0	1.99
		0.2	1.66
		0.1	1.36
		0.05	1.12
5.0	Single hinge at base	1.0	3.09
	Multiple hinges	1.0	2.53
		0.2	2.20
		0.1	1.84
		0.05	1.40

Two important questions are what reduction of shear rigidity is appropriate for a particular wall, and is it appropriate to distribute the shear deformations over the full height of the wall by using a reduced effective shear rigidity over the full height of the wall? To answer these and other questions, a nonlinear hysteretic shear model was developed based on the results from cyclic shear testing of reinforced concrete membrane elements, and this nonlinear shear model was used to analyze the current example (Rad, 2008). The results from this additional work indicate that an effective shear rigidity of 20% $G_c A_{vg}$ over the full height of the wall results in a very good estimate of the dynamic shear magnification.

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