EVALUATION OF REINFORCED CONCRETE CRACKING AND POST-CRACKING BEHAVIOR FOR NUCLEAR BUILDINGS UNDER DESIGN AND BEYOND DESIGN LEVEL SEISMIC INPUT MOTIONS

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ABSTRACT

The paper presents an efficient nonlinear SSI approach for evaluating the reinforced concrete wall cracking and post-cracking behaviour in nuclear structures. The nonlinear SSI approach is based on a hybrid approach that uses iterative equivalent-linearized models for the concrete walls (panels) that correspond to the strain levels in different parts of the concrete structure. The hybrid approach is both accurate and extremely fast since convergence is achieved in only few iterations that are restart SSI analysis runs. This hybrid approach was implemented in the ACS SASSI Option NON software. Based on the ASCE 4-16 standard recommendations for the site-specific license applications, the hybrid nonlinear SSI approach based on the equivalent-linearization is applicable to both: i) Design level (DBE) applications for evaluation of the reinforced concrete cracking pattern in structures as a function of in-plane wall strains and ii) Beyond design level (BDBE) applications for evaluation concrete post-cracking behaviour for computing the safety margins or the structural and equipment fragilities. A typical low-rise concrete shearwall nuclear building was considered. The nonlinear SSI results are compared without and with considering the ASCE 4-16 standard recommendations for limiting the material damping values for the Response Levels 1, 2 and 3 to 4%, 7% for DBE and 10% for BDBE, respectively.

1. INTRODUCTION

The nonlinear hysteretic model library in ACS SASSI Option NON (2016) for simulating nonlinear reinforced concrete wall behavior includes three models: i) Cheng-Mertz shear (CMS) model, ii) Cheng-Mertz bending (CMB) model, and iii) Takeda model (TAK). The Cheng-Mertz hysteretic model was used over years in a number of studies for the DOE and ASCE standards. Figure 1 shows a comparison of the three hysteretic model behaviour under a random displacement history (upper plots) and a single cycle history (lower plots), respectively, for two BBC with different concrete cracking levels. The cracking point on the BBC are marked by a black dot.

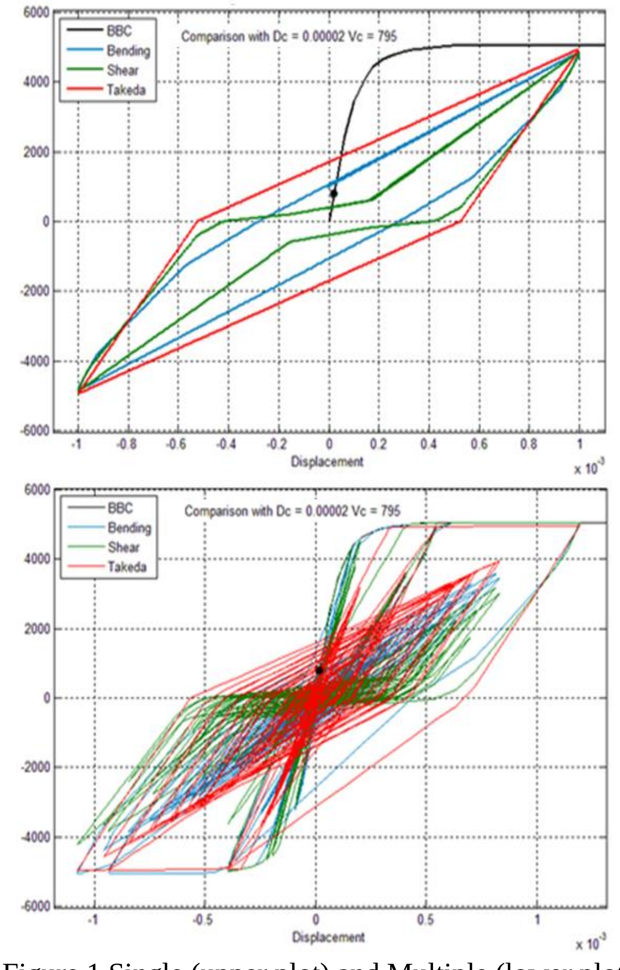

Figure 1 Single (upper plot) and Multiple (lower plot) Hysteretic Loops for CMS, CMB and TAK Models

It should be noted that the CMS hysteretic model in contrast to the CMB and TAK model has the capability to capture well significant shear stiffness degradation for the larger loading cycles (similar to an origin-oriented hysteretic model), but also to capture the reduced stiffness degradation for the unloading cycles and the pinching effects which occur for low amplitude cycles as shown in Figure 1.

However, before doing the nonlinear SSI analysis, the structural model and walls need to be divided in “panels” for which the assumption of the uniform shear or bending deformation is applicable as shown in Figure 2. Figure 2 shows the external view of the nonlinear structure model split in 40 wall panels (with no roof, basemat and longitudinal external wall.

This model split in "panels" can be done by the analyst when the FE model is generated, or later using the powerful set of automatic ACS SASSI user-interface (UI) commands, such as WALLFLR, SPLITWALLS, SEGWALLS, PANELGEN, BBCGEN, etc. Thus, generating the nonlinear model is a rapid user-friendly process to build the FE models of nuclear buildings with plane vertical walls and horizontal floors. The UNIPL command is capable of handling curved concrete walls such as containment shells which need to be split at the shell element level.

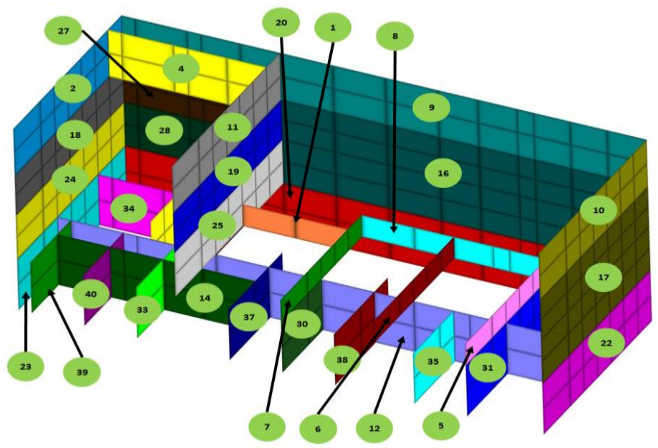

Figure 2 Shearwall Building Model Split in Panels

Based on the in-plane hysteretic behaviour of each wall panel in the time-domain, the local equivalent-linear panel properties are computed at each SSI iteration performed in the complex frequency-domain. The stiffness reduction is applied directly to the elastic modulus for each panel. This implies, under the isotropic material assumption, that the shear, axial and bending stiffness suffer the same level of degradation. Poisson ratio is considered constant. Thus, in the current implementation, the wall panel shear stiffness modification as a result on nonlinear behaviour is fully coupled with the bending stiffness. This is a reasonable assumption *only* for the low-rise shearwalls for which the nonlinear behaviour is governed by the shear deformation. Based on various experimental tests done at the Cornell University, Gergely states in NUREG/CR 4123 (Gergely, 1984) that "in low-rise walls such as those that occur in the modern nuclear power plants, the flexural distortions and associated vertical yielding play a negligible role." This was also recognized by other research studies, including the EPRI report on "Methodology for Developing Seismic Fragilities" (Reed and Kennedy, 1994).

After splitting the nonlinear model into wall panels, the nonlinear behaviour of each panel has to be defined by its back-bone curve (BBC) and hysteretic type model.
The BBC for each panel depends on the panel geometry, thickness, concrete and reinforcement strengths and ratios. For each panel, analysts need to define the BBC curve. The BBC should be built based on the concrete cracking and ultimate wall capacities under either shear or bending deformation. The shear and bending strains in wall are determined based on the panel corner relative displacements after the rigid body transformation is subtracted. It should be noted that the horizontal and vertical displacements computed at the panel corners of each wall must include the combined effects of the three seismic component inputs.

For estimating the low-rise shearwall panel capacities there are a significant number of pertinent sources in the literature that provide different empirical equations for computing the wall panel shear capacities (Gulec and Whittaker, 2009). Using the SHEAR command the user can check the computed shear capacity values based on different shear capacity equations (Ghiocel and Saremi, 2017).

Further, using the BBCGEN command, smooth BBC curves can be automatically generated for many wall panels. Before using the BBCGEN command, the user needs to decide which of the four shear ultimate capacity models of the SHEAR command he would like to consider for the nonlinear SSI analysis. The smoothed BBC are automatically generated for all wall panels based on the cracking and ultimate shear force values and assuming that the secant cracked stiffness between the cracking and yielding points is half of the uncracked stiffness as recommended in the ASCE 4-16 standard, and USNRC SRP 3.7.2. Details on the BBC construction based of the wall shear capacities and application of the BBCGEN and SHEAR commands are provided in a previous paper (Ghiocel and Saremi, 2017).

2. ASCE 4-16 RECOMMENDATIONS

The ASCE 4-16 Section C3.3.2 recommends for the evaluation of the concrete cracking pattern for the site-specific applications, to use at the least a two-step procedure as described here. First, the linear SSI analysis is performed for the uncracked structure, to compute the stresses in structure walls. If the wall shear stress is larger than $3\sqrt{f'_c}$ (or shear strain larger than $3\sqrt{f'_c}/G_c$), or the wall bending stress is larger than $7.5\sqrt{f'_c}$ (or bending strain larger than $7.5\sqrt{f'_c}/E_c$), then, the concrete wall is considered fully cracked, so that its stiffness goes down to 50% of elastic stiffness, and its damping goes up to 7%. After, the concrete wall properties are changed accordingly in the structure model, the second SSI analysis is performed using the linearized partially cracked model to obtain the final SSI results. The ASCE 4-16 standard Section C3.3.2 states that *"After running the second analysis that includes cracked properties for some or all walls, rechecking the wall stress state is not necessary.*" In other words, the ASCE 4-16 standard considers that performing only 1 iteration equivalent-linear SSI analysis is reasonable accurate.

To be in full compliance with the ASCE 4-16 standard recommendations for the design-level analyses while running the ACS SASSI Option NON, the analyst is required to define a cut-off damping value of 7%, so that computed material damping values for the concrete walls are not allowed to go higher than 7%. However, from a theoretical point of view, the nonlinear SSI analysis should be performed without any damping cut and run SSI analysis iteratively until the convergence is fully reached. The convergence is achieved typically in about 3-4 SSI iterations for the design-level and about 4-6 SSI iterations for beyond design-level. The SSI iteration restart runtimes are typically about 40-50% of the initial elastic SSI analysis runtime.

For DBE level, the ASCE 4-16 standard (and ASCE 43-18 draft) defines the Response Levels 1, 2 and 3 based on the seismic stress-levels in the concrete walls. For Response Level 1, the uncracked concrete stiffness and damping are used, while for Response Level 2, the cracked concrete reduced stiffness by 50% for shear and bending stiffnesses and increased damping values of 7% are acceptable. The ASCE recommendations that limit the damping increase to 7% are aimed to introduce an additional level of conservatism in the seismic analysis process.

For BDBE levels, the ASCE 4-16 standard recommendations for the cracked concrete walls at the Response Level 3 is to limit the damping at 10%. For the effective concrete wall stiffness, there is no precise recommendation on the effective stiffness decrease at the Response Level 3.

3. DESIGN-BASIS (DBE) LEVEL

A typical low-rise concrete shearwall nuclear building was used. Figure 2 shows the external view of the nonlinear structure model split in 40 wall panels (left plot) with no roof, basemat and longitudinal external wall. The wall panels are identified by different colours. Figure 3 also shows on the FE structure model (right plot) the locations of interest for computing ISRS, Node 570 at lower elevation and Node 143 at higher elevation, and the Panel 17 that is the transverse external wall that shows the largest nonlinear behaviour comparing with other wall panels.

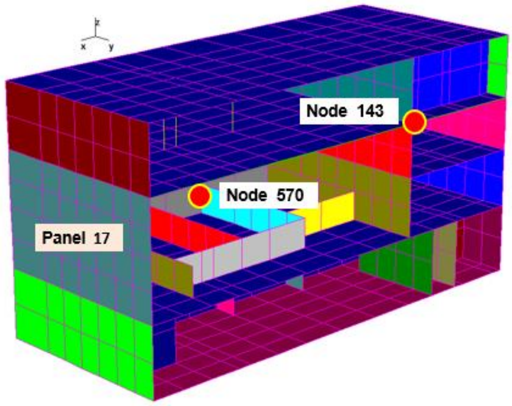

Figure 3 Selected SSI Response Locations

The soil deposit was idealized by a uniform rock media with a shear wave velocity of 5,000 fps. The nonlinear structure model includes a total of 40 wall panels. The BBC for all wall panels were automatically developed based on the ultimate shear capacity computed using the Wood empirical equation (Gulec and Whittaker, 2009) and the ASCE 4-16 Section C3.3.2 cracking criteria based on the local shear stress level. The reinforced concrete wall nonlinear behaviour was idealized by the Cheng-Mertz shear (CMS) hysteretic model. The DBE seismic input was defined by the RG1.60 spectrum anchored at 0.30g maximum ground acceleration.

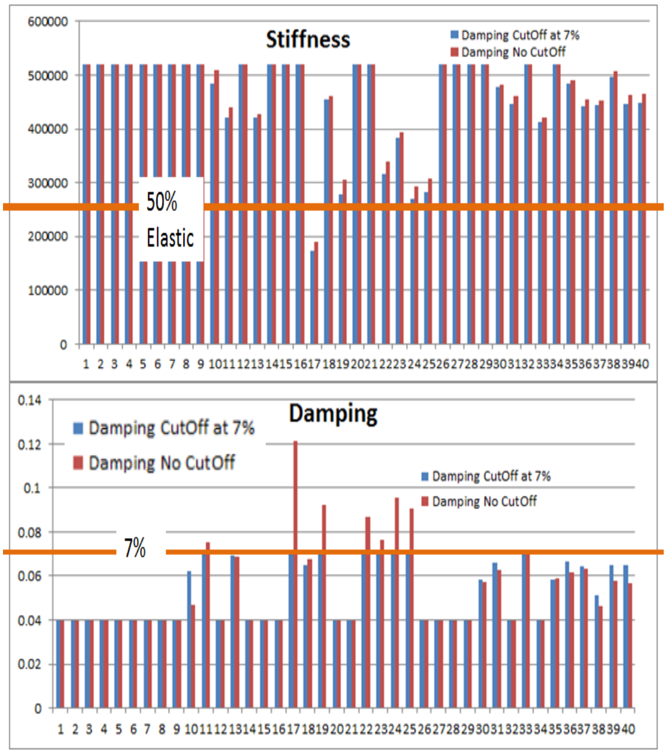

Figure 4 Equivalent-Linear Wall Panel Stiffness and Damping Values For the 0.30g DBE Seismic Input

The ASCE 4-16 7% damping cut-off value can be automatically considered by the analyst in the ACS SASSI Option NON. A comparison between the nonlinear SSI analysis results obtained using 7% cut-off damping value for concrete walls per the ASCE 4-16 recommendations and the nonlinear SSI solution with no damping cut-off is illustrated in next section.

Figure 4 shows the computed effective stiffness and damping for all 40 wall panels. It should be noted that the effect of introducing the conventional 7% damping cut-off as recommended by seismic design regulations has a small impact on the effective wall stiffness values, and more significant on the damping values. The main transverse walls are the Panels # 17, 19, 22-25 (for precise locations of the wall panels, please see Figure 2) between 2nd and 4th floors. These walls indicate a significant concrete cracking, as their effective stiffness values drop to 40%-65% of the initial uncracked concrete stiffness. These transverse walls have also larger hysteretic damping values than the 7% damping value, up to 12% for the Panel 17 that is the most seismically loaded wall. It can be remarked that for the transverse walls between the 1st and 2nd floors, the stiffness reduction is considerably less, being not more than 15%-20%, since at this level there is a large number of transverse walls.

The effects of applying the "conventional" 7% cut-off damping value (blue line), as required by the ASCE standards and USNRC guidelines, on the structure nonlinear hysteretic SSI response is shown in Figure 5 for the Panel 17 story drift.

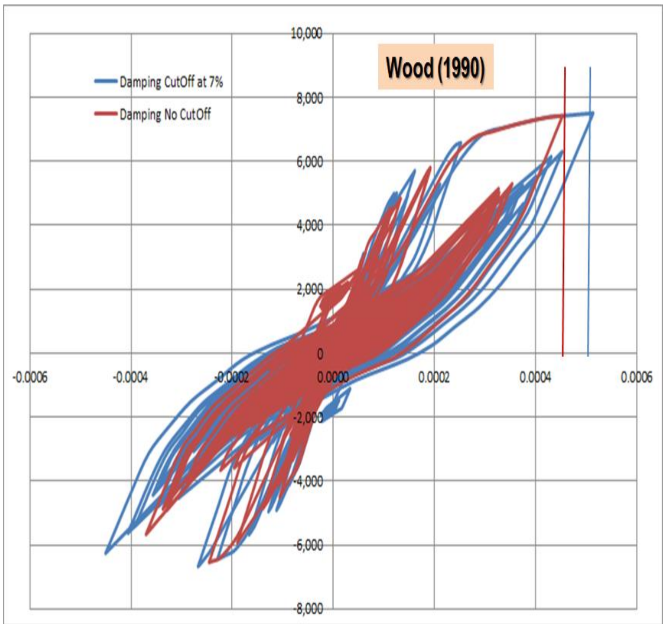

Figure 5 Panel 17 Hysteretic Loops With No Damping Limitation and With 7% Damping Limit

It can be seen that the 7% damping cut-off increases the wall drift response by only 10% in comparison with the nonlinear SSI analysis solution with no damping cut-off.

Figure 6 shows the in-structure response spectra (ISRS) at a lower and a higher elevation in the structure. The two locations that correspond to node 143 (higher elevation) and node 570 (lower elevation) are indicated in Figure 3. It should be noted that the effect of concrete cracking affects significantly the ISRS results. The reduction of the ISRS peaks is about 40% for the higher elevation ISRS and about 20% for the lower elevation ISRS. It should be also remarked that the nonlinear response ISRS computed without and with the 7% damping cut-off have close values with differences of about 5%. This shows again that the impact of introducing the damping upper limit at 7% as required by the ASCE 4-16 standard is minimal.

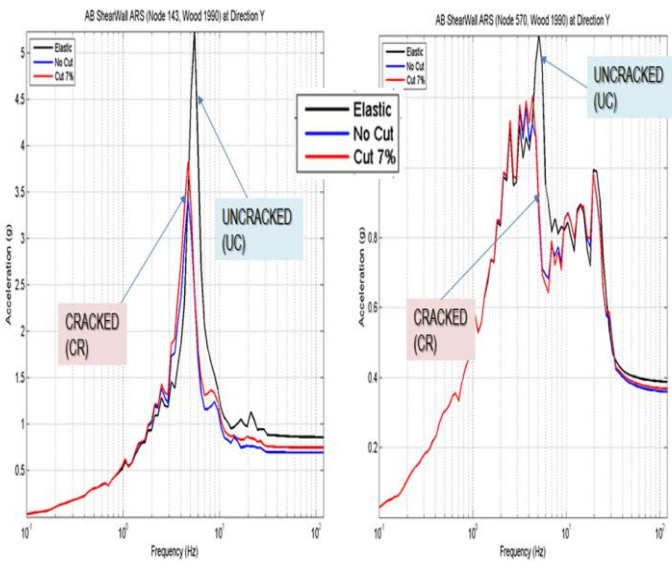

Figure 6 Effects of Damping Limits on the ISRS at Higher (Node 143) and Lower (Node 570) Elevations

Figure 6 shows that for the investigated structure and for design-basis level (DBE), the concrete cracking effects produces lower ISRS in comparison with uncracked concrete ISRS. However, the resonant peak frequency shifts due to the structure stiffness reduction are relatively small (for the node 143 ISRS plots, from about 6.3 Hz for uncracked model to 5.0 Hz for cracked model).

4. BEYOND DESIGN-BASIS (BDBE)

The same building model and soil deposit were used. The BDBE seismic input was defined by the RG1.60 spectrum anchored at 0.60g maximum ground acceleration (twice than DBE level).

Figure 7 illustrates the effective stiffness and damping values for all wall panels for three cases, specifically, the nonlinear SSI analysis with no damping limit as the "reference" approach and the nonlinear SSI analysis with the 7% and 10% damping limits, respectively, as "conventional" approaches. The 7% damping limit in Response Level 2 is used here although per the ASCE 4-16 standard this damping corresponds to the cracked concrete walls for which the local stresses are between 50% to 100% of wall yield capacity which are usually too low for the beyond design-level input conditions. The ASCE 4-16 standard recommends the use of the Response Level 3 damping limit of 10% for the beyond design-basis level (BDBE) seismic analysis, apparently independently of the beyond design review level.

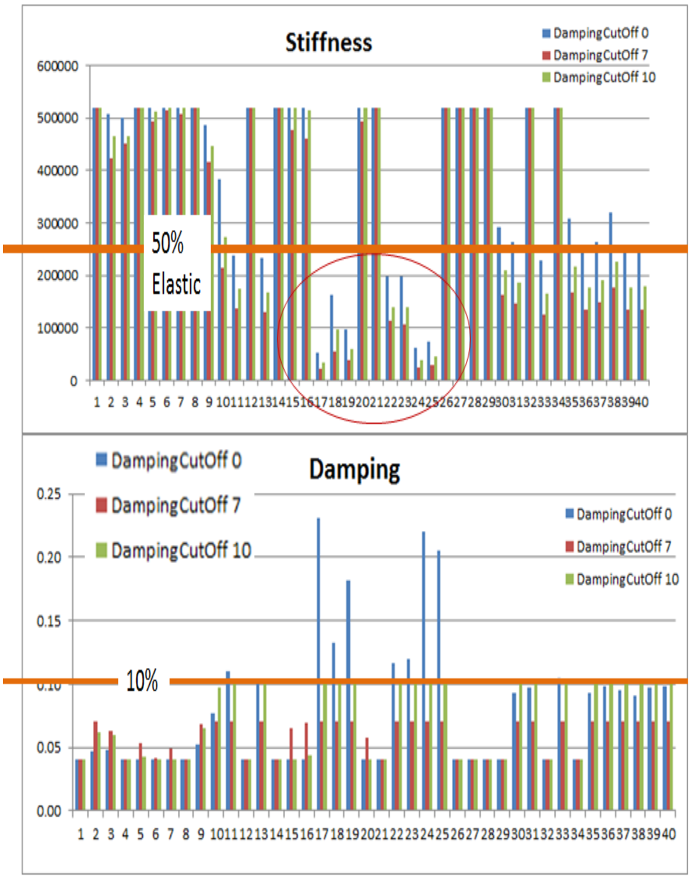

Figure 7 Equivalent-Linear Wall Panel Stiffness and Damping Values for the 0.60g BDBE Seismic Input

One aspect that strikes attention in Figure 7 are the large reductions of the effective wall stiffness values in the transverse direction, shown in the red circle, including the Panels 17, 19, 22-25 between 2nd and 4th floors which see significant damages (Panels 20 and 21 are longitudinal walls).

The effective wall stiffness values are severely below the 50% reduced stiffness used in practice for cracked concrete. The worst case is Panel 17 that sees an effective stiffness reduction of about 8-10 times in comparison with the initial uncracked wall stiffness. The effect of limiting the damping value to 10% for Response Level 3 is significant in this case. The computed wall stiffness reductions can be double if the concrete damping values are limited per ASCE 4-16 recommendations.

As a result of the large wall stiffness reductions, concrete structure dynamics could change quite drastic and this can affect largely the nonlinear SSI responses, especially the ISRS shapes. This a serious modelling issue for performing meaningful fragility analysis based on probabilistic nonlinear SSI analysis (Ghiocel, 2016, 2017). Another remark is that the transverse walls and the longitudinal walls are damaged quite differently. As expected, the longitudinal walls that have much larger web lengths and, therefore, much larger wall stiffness and capacities, are much less damaged.

Figure 8 shows that the shear force-shear strain hysteretic behaviour of the Panel 17. It should be noted that using a 10% damping cut-off value produces an increase of the nonlinear story drift of about 50% in comparison with the reference nonlinear SSI analysis with no damping cut-off.

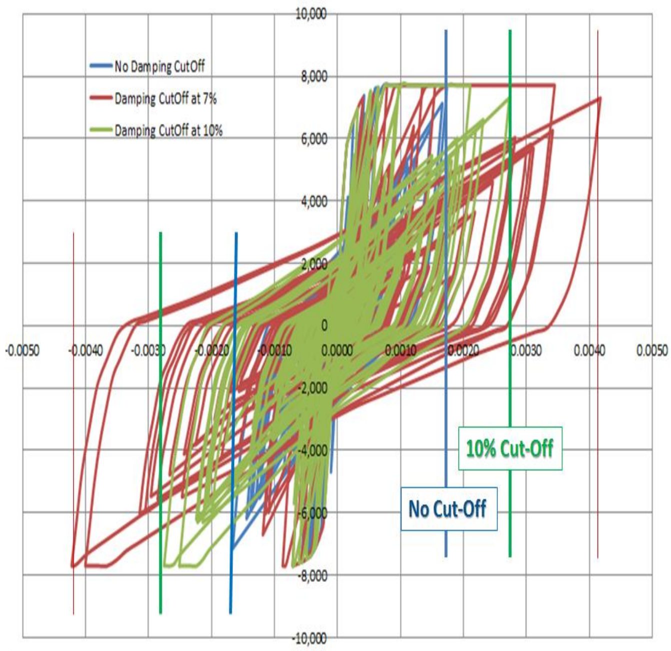

Figure 8 Panel 17 Hysteretic Loops With No Damping Cut-Off and With 10% Damping Cut-Off for the 0.60g BDBE Seismic Input

The SSI response overestimation in Figure 8 for the 10% damping cut-off response indicates a limitation of the application of the ASCE 4-16 standard guidance for the Response Level 3 for the BDBE applications.

Figure 9 shows the effects of ASCE 4-16 recommendations on the damping limits on the seismic ISRS. The same two ISRS locations were considered. The effects of the damping limitation are large, especially for the ISRS with a narrow band frequency content, typically occuring for the high elevation ISRS. For the ISRS at the high elevation (node 143), the 10% damping cut-off produces an increase of 60% of the spectral peak amplitude. However, for the ISRS at the low elevation (node 570), the ISRS peak amplitude increase is only about 10% for the 10% damping cut-off limit.

It should be also noted that the nonlinear SSI results produces totally different ISRS in comparison with uncracked concrete linear SSI analysis. The significant shifts in the structural resonant frequencies due to the large wall stiffness reductions in a number of shearwalls, especially transverse walls, change completely the ISRS peaks and even shapes as shown in Figure 9.

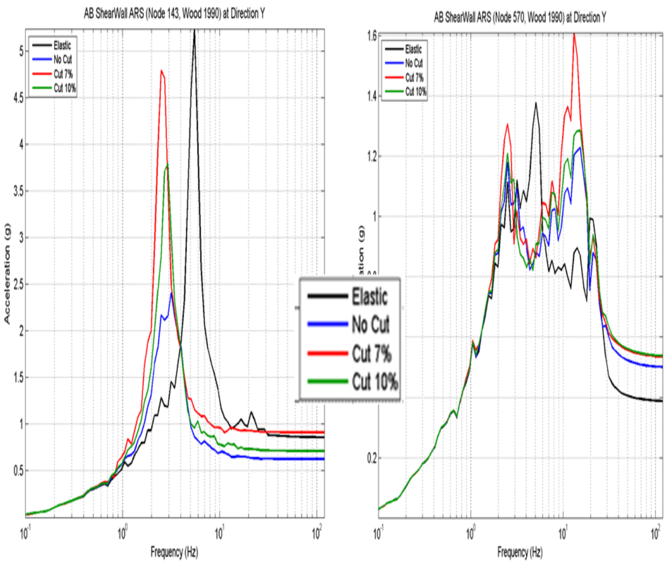

Figure 9 Effects of Damping Limits on the ISRS at Higher (Node 143) and Lower (Node 570) Elevations

Figure 9 results show that nonlinear behavior could produce much lower, but also much larger ISRS peaks, as it happened in the 2-3 Hz range for node 143 ISRS (left), about 3.75 times larger, and in the 12-15 Hz range for node 570 ISRS (right), about 1.5 times larger.

The comparative ISRS results shown in Figure 9 indicate that the application of the nonlinear SSI hybrid approach to the BDBE seismic analyses is of key importance to obtain reasonable ISRS estimates and to perform meaningful structure and equipment seismic margin and fragility analysis evaluations.

5. CONCLUSIONS

The paper illustrates the application of an efficient nonlinear SSI hybrid approach based on an iterative equivalent-linearization procedure to the seismic analysis of nuclear concrete structures in accordance to the ASCE 4-16 recommendations.

A low-rise shearwall building is used to demonstrate the nonlinear SSI hybrid approach and interpret its results in the light of the ASCE 4-16 standard recommendations for both the design-basis (DBE) level and beyond the design-basis level (BDBE).

6. REFERENCES

ASCE 4-16 Standard (2017). "Seismic Analysis for Safety-related Nuclear Structures and Commentary", *American Society of Civil Engineers, Washington, D.C.*

Barda, F., J.M. Hanson, and G.W. Corley (1977) "Shear Strength of Low-Rise Walls with Boundary Elements", *ACI Special Publication SP 53-8.*

Cheng, Y.F. and Mertz, G. (1989)."Inelastic Seismic Response of Reinforced Concrete Low-Rise Shear Walls of Building Structures", *Univ. of Missouri-Rolla, Dept. of Civil Engineering*, CE 89-30

Gergely, P.(1984) "Seismic Fragility of Reinforced Concrete Structures and Components for Application in Nuclear Facilities", *Cornell University, Published as NUREG/CR-4123*

Ghiocel, D.M. and Saremi, M. (2017). "Automatic Computation of Strain-Dependent Concrete Cracking Pattern for Nuclear Structures for Site-Specific Applications", *SMiRT24, Busan, Korea, August 20-25*

Ghiocel D.,M. (2015) "Fast Nonlinear Seismic SSI Analysis Using A Hybrid Time-Complex Frequency Approach For Low-Rise Nuclear Concrete Shearwall Buildings", the *SMiRT22 Conference Proceedings, Division V, Manchester, UK, August 10-14*

Ghiocel Predictive Technologies, Inc. (2018). "ACS SASSI - An Advanced Computational Software for 3D Dynamic Analyses Including SSI Effects", *ACS SASSI Version 3 User Manuals, February 28*

Gulec, C.K. and A.S. Whittaker (2009). "Performance-Based Assessment and Design of Squat Reinforced Concrete Shear Walls", *Technical Report, MCEER, No. 09-0010*