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FAST NONLINEAR SEISMIC SSI ANALYSIS OF LOW-RISE CONCRETE SHEARWALL BUILDINGS FOR DESIGN-LEVEL (DBE) AND BEYOND DESIGN-LEVEL (BDBE)

Dan M. Ghiocel[1] YoungSun Jang[2] and InHee Lee[3]

[1] President, Ghiocel Predictive Technologies, Inc., New York, USA (dan.ghiocel@ghiocel-tech.com)
[2] Professor, KEPCO E&C, Gimcheon, Korea (ysjang63@naver.com)
[3] President of CNG Softek Co. Ltd., Seoul, Korea (leeih@cngst.com)

INTRODUCTION

The paper illustrates the application of the fast nonlinear SSI approach to reinforced concrete nuclear buildings. The fast nonlinear SSI approach is based on a *hybrid approach* that couples the global linearized SSI analysis solution in the complex frequency domain with a local nonlinear analysis solution in the time domain (Ghiocel, 2015). The hybrid approach uses an efficient iterative equivalent-linearization procedure that convergences in only 2-4 iterations for the design-level and 4-8 iterations for the beyond-design level. The nonlinear SSI approach was implemented in the ACS SASSI Option NON software (2016). The nonlinear SSI analysis based on the hybrid approach is only 2-3 times slower than a linear SSI analysis, and likely hundreds of times faster than a *true* nonlinear SSI analysis based on the time-integration that requires that a significant part of the surrounding soil media to be included in the FE model together with the structure.

The nonlinear SSI analysis based on the hybrid approach is demonstrated for a low-rise reinforced concrete shearwall nuclear building. The nonlinear SSI results are compared without and with considering the ASCE 4 and 43 standard recommendations for limiting the material damping values for the Response Levels 1, 2 and 3 to 7% and 10%, respectively. It should be also that using the fast nonlinear SSI approach, the stress-dependent concrete cracking pattern in a structure can be automatically identified for the design-basis level (Ghiocel and Saremi, 2017). Herein, the fast nonlinear SSI analysis based on the hybrid approach is applied to a concrete shearwall building for both the seismic design-basis level input (DBE) and the beyond design-basis level input (BDBE). As a special case, the fast nonlinear analysis is also illustrated for a static pushover analysis of a concrete containment structure (CS). The pushover analysis is shown as an example on how to use the CS experimental global pushover test results to calibrate the nonlinear force-displacement relationship or back-bone curve (BBC) for each shell element in the CS cylinder to be used sequentially for the nonlinear seismic SSI analysis.

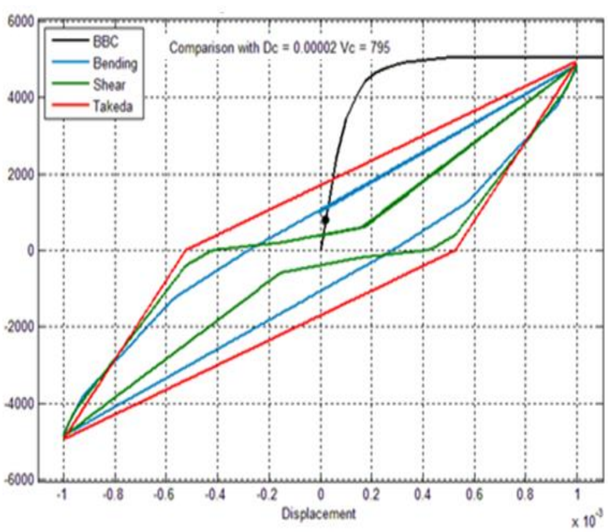
NONLINEAR CONCRETE BEHAVIOR MODELING

In the hybrid SSI approach, the local equivalent-linear properties of the concrete walls are computed repeatedly based on the hysteretic behaviour of each local wall in the time-domain at each SSI iteration until the convergence is reached. The stiffness reduction is applied directly to the elastic modulus for each wall panel. This implies, under the isotropy material assumption, that the shear, axial and bending stiffness suffer the same level of degradation. Poisson ratio is considered to remain constant. Thus, in the current implementation, the wall panel linearized stiffness modification as a result on nonlinear behaviour is fully coupled for the shear and the bending stiffnesses. 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). However, if large openings are present in the walls, the walls should be split in a number of pier and spandrel wall panels. It should be noted that the bending strains could become dominant for higher height-to-width ratio wall piers. The analyst can handle this by selecting the bending deformation as the controlling deformation mechanism than the shear deformation. Both the shear and the bending deformations can be considered separately to investigate preliminary the governing deformation mechanisms for selected wall panels before the final nonlinear modelling is decided.

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 assuming either shear or bending deformation as governing nonlinear behavior mechanism (Ghiocel and Saremi, 2017). It should be noted that the horizontal and vertical in-plane wall strains computed must include the combined effects of the three seismic component inputs, since the linear superposition of the co-directional effects is not permissible due to the nonlinear behavior.

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.



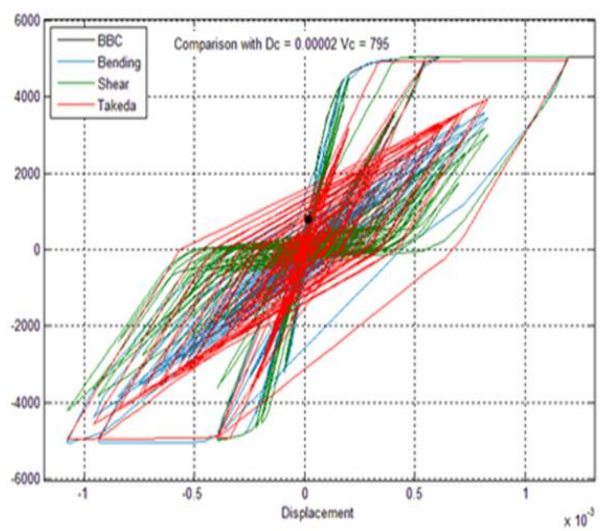


Figure 1 Single (left) and Multiple (right) Hysteresis Cycles for CMS, CMB and TAK Models

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. Details on the BBC construction based of the wall shear capacities, elastic stiffness and concrete cracking level are provided in the ACS SASSI Option NON documentation and also in Ghiocel and Saremi, 2017.

LOW-RISE SHEARWALL BUILDING CASE STUDY

A typical low-rise concrete shearwall nuclear building was used. To build the nonlinear SSI model, the structure walls are subdivided in a number of "panels" for which the assumption of the uniform shear or bending deformation assumption is reasonable. This can be done by the analyst when the FE model is built, but also later, using the specialized set of automatic ACS SASSI user-interface (UI) commands. 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 2 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.

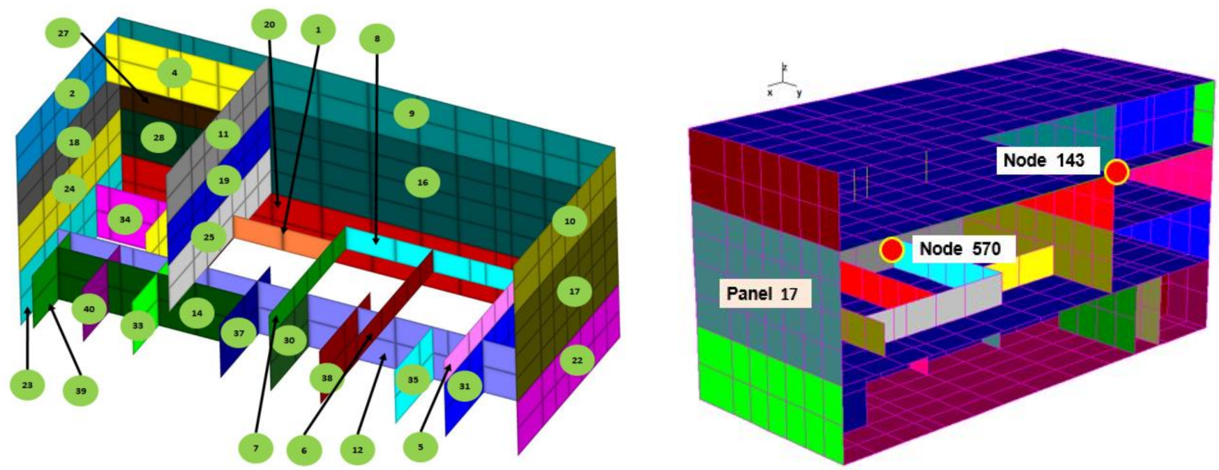


Figure 2 Single Material Linear Structure Model (Left Plot) and Uncovered Multi Material Nonlinear Structure Model (Right Plot)

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-1999 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 seismic input was defined by the RG1.60 spectrum anchored at 0.30g maximum ground acceleration for the design level and 0.60g for the beyond-design level. The design level and beyond-design level nonlinear SSI analysis results are shown in Figures 3 thru 12.

0.30g Design-Level (DBE) Including Response Levels 1 and 2

The ASCE 4 and 43 standards define 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. In this section, the results of a nonlinear SSI analysis with no damping limit, considered as a the "reference" approach from theoretical-basis point of view, are compared with results of the nonlinear SSI analysis with 7% damping limit per the ASCE recommendations, as "conventional" approaches. The

ASCE damping limits can be automatically considered by the analyst in the ACS SASSI Option NON software.

Figure 3 shows the computed effective stiffness and damping for all 40 walls for the 0.30g design-level seismic input. 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 4 for the Panel 17 story drift. 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.

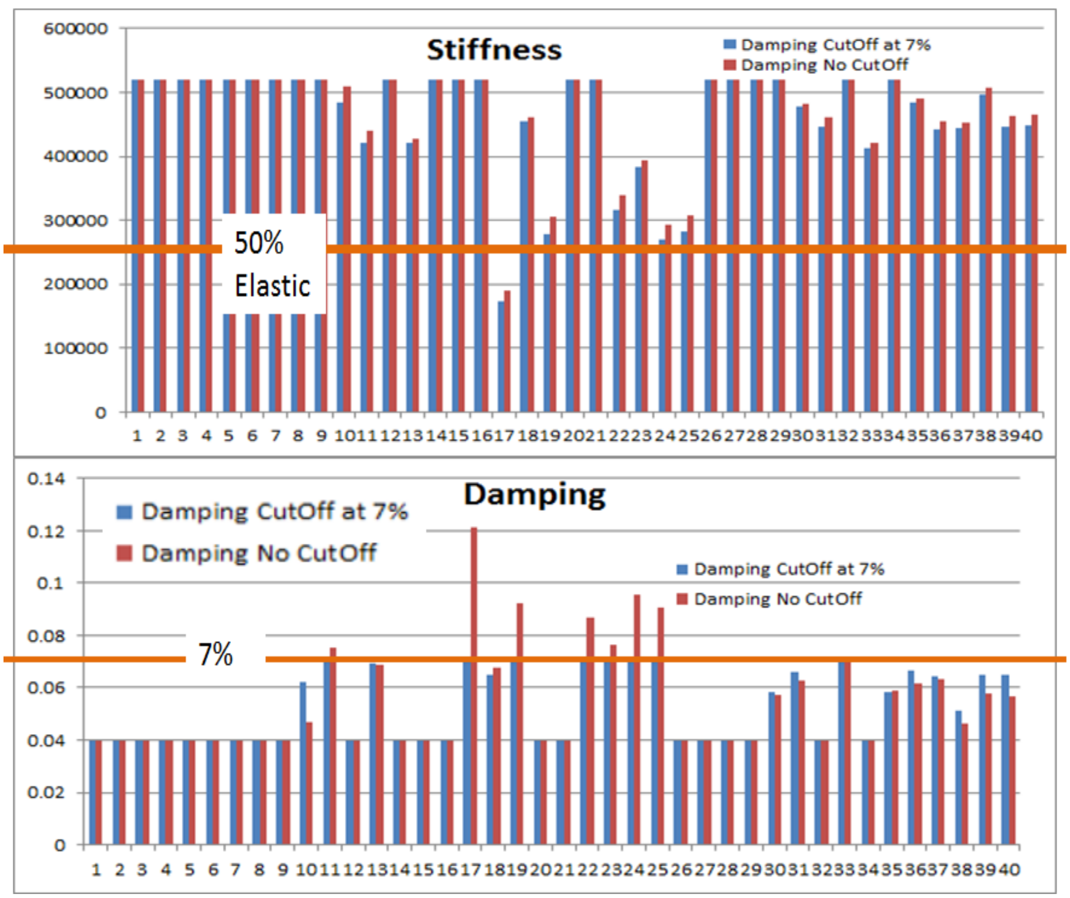


Figure 3 Effective (Equivalent-Linear) Panel Stiffness and Damping Values
For the 0.30g Design-Level Seismic Input

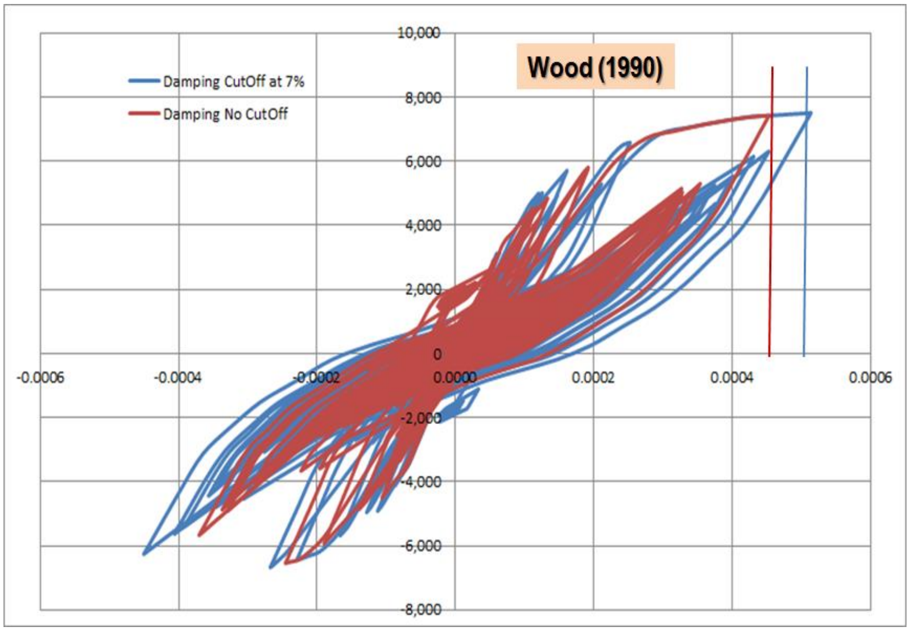


Figure 4 Panel 17 Hysteretic Loops With No Damping Limitation and With 7% Damping Limit
For the 0.30g Design-Level Seismic Input

Figure 5 shows the in-structure response spectra (ISRS) at a lower and a higher elevations in the structure. The two locations that correspond to node 143 (higher elevation) and node 570 (lower elevation) are indicated in Figure 2. 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 design codes is minimal.

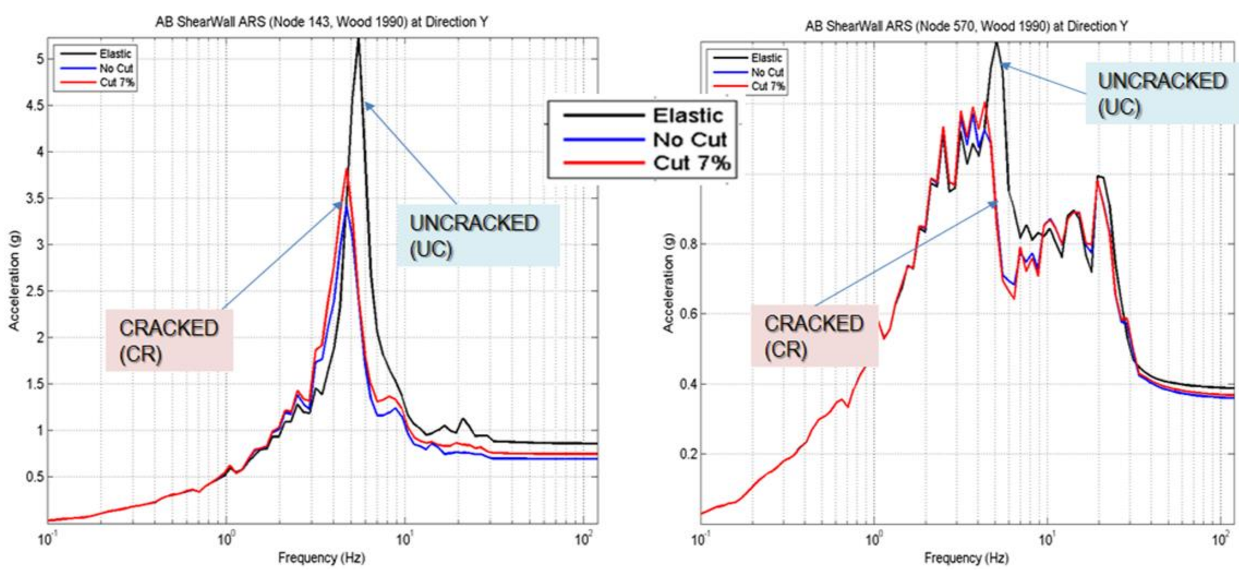


Figure 5 Effects of Damping Limits on the ISRS at Higher (Node 143) and Lower (Node 570) Elevations

Figure 5 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).

0.60g Beyond Design Level (BDBE) Including Response Levels 2 and 3

The ASCE standard recommendations for the cracked concrete walls at the Response Levels 2 and 3 is to limit the damping at 7% and 10% respectively. For the effective concrete stiffness, a 50% reduction is acceptable for the shear and bending wall stiffnesses at the Response Levels 2. There is no precise recommendation on the effective stiffness decrease at the Response Level 3.

The nonlinear SSI analysis results obtained for the beyond design-level are shown in Figures 6 thru 12. Figure 6 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.

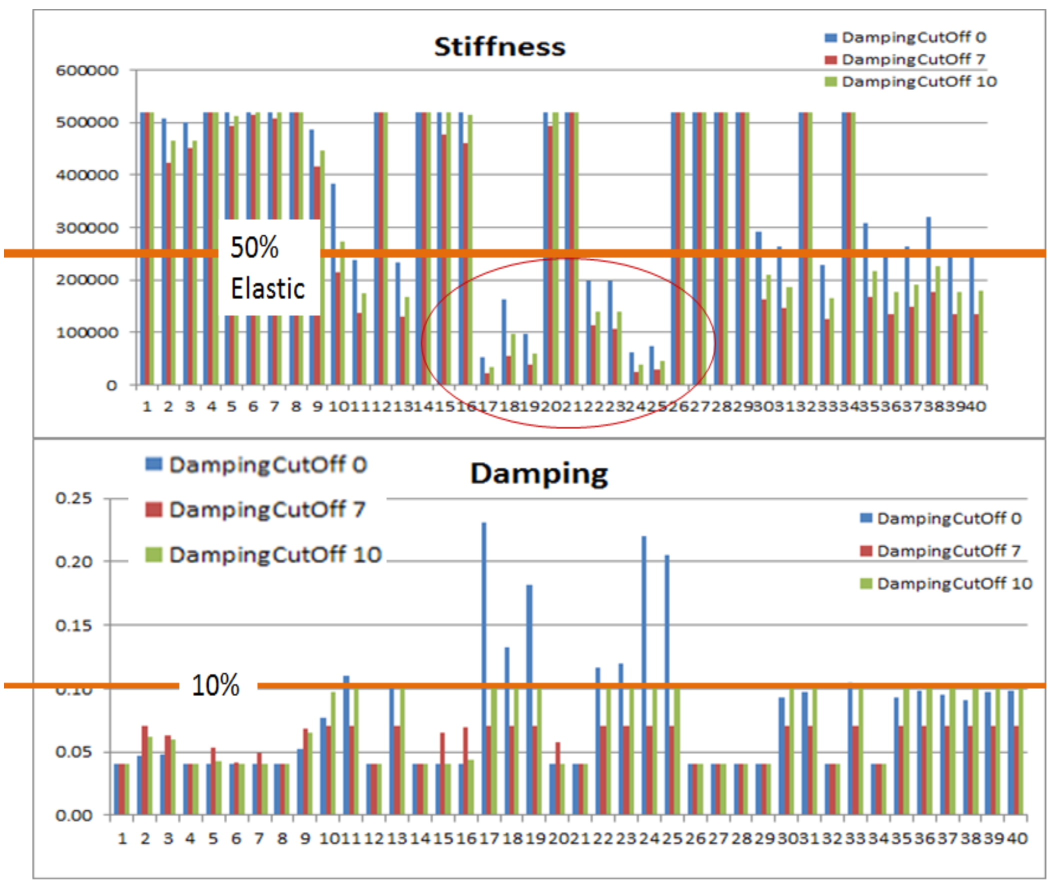


Figure 6 Effective (Equivalent-Linear) Wall Panel Stiffness and Damping Values
For the 0.60g Beyond Design-Level Seismic Input

One aspect that strikes attention in Figure 6 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 10 times in comparison with the initial uncracked stiffness. The effect of limiting the damping values to 7% and 10% is significant. The computed wall stiffness reductions can double if the concrete damping values are limited per ASCE 4-16 recommendations. As a result 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 different. As expected, the longitudinal shearwalls that have much larger web lengths and, therefore, much larger wall stiffness and capacities, are much less damaged.

Figure 7 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 “conventional” damping limits. The use of the 7% damping cut-off value produces an increase of the story drift of more than 250%, and by this largely exaggerates the wall stiffness reduction which has an immediate impact on the overall structure dynamics, as can be seen in Figure 8. This indicates the limitation of the application of the ASCE 4-16 standard simple guidance for the Response Level 3 for the beyond design-basis level (BDBE) applications.

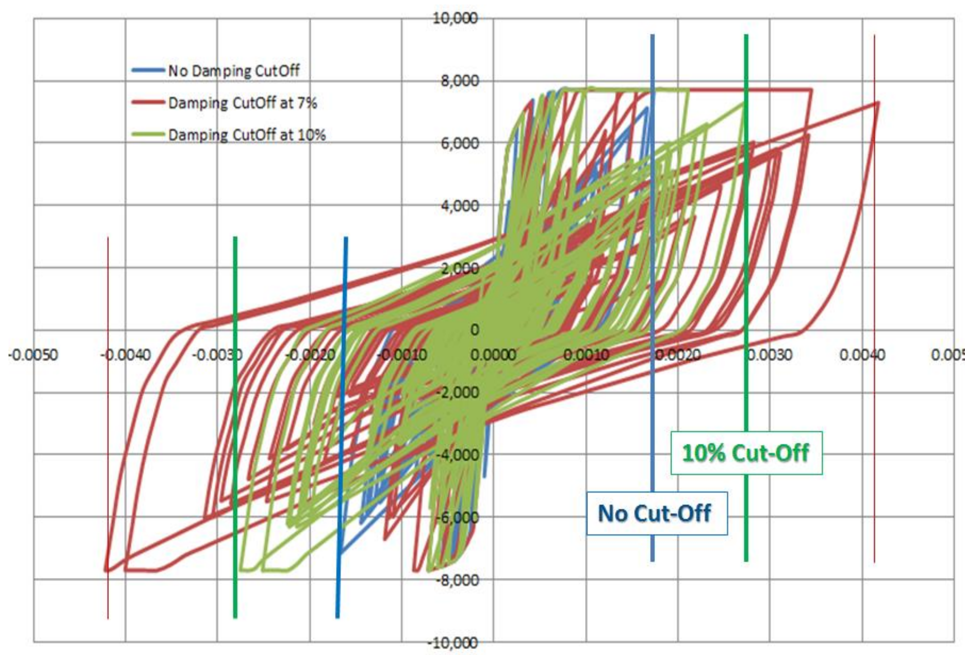


Figure 7 Panel 17 Hysteretic Loops With No Damping Limitation and With 7% and 10% Damping Limits For the 0.60g Beyond Design-Level Seismic Input

Figure 8 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 limit produces an increase of 60% of the spectral peak amplitude, while the 7% damping limit produces an increase of about 200%. However, for the ISRS at the low elevation (node 570), the ISRS peak amplitude increase is only 20-30% for the 7% damping limit.

It should be 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 8.

Figure 8 results show that large nonlinear behavior could produce much lower, but also much larger ISRS peaks, as it happened in the 2-3 Hz range for the node 143 ISRS (left), about 3.75 times larger, and in the 12-15 Hz range for the node 570 ISRS (right), about 1.5 times larger.

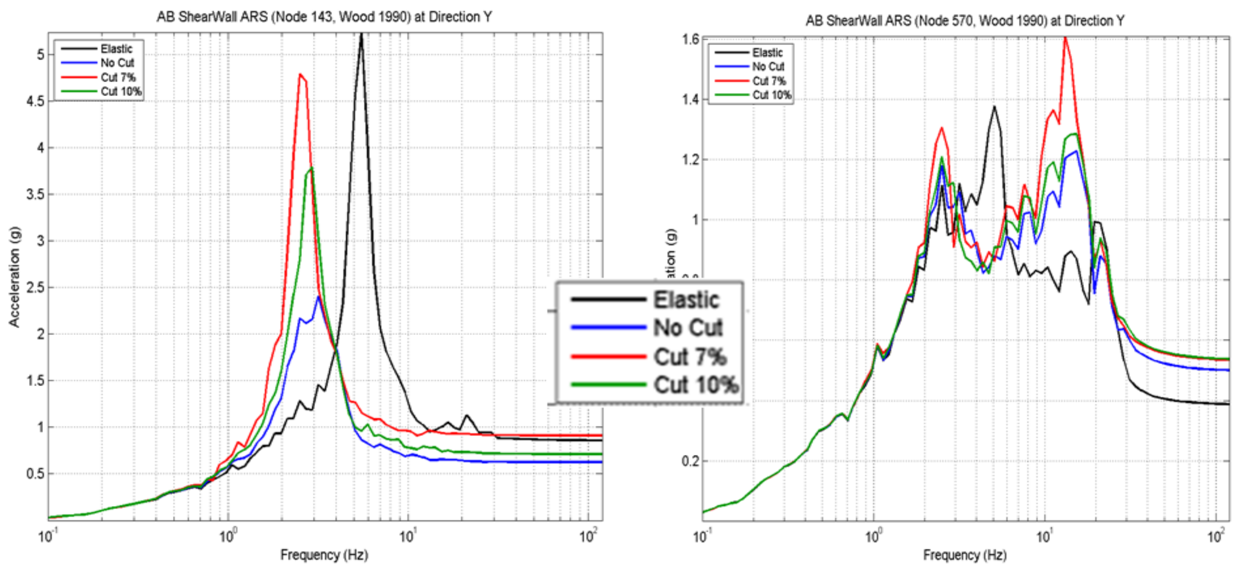


Figure 8 Effects of Damping Limits on the ISRS at Higher (Node 143) and Lower (Node 570) Elevations

The comparative ISRS results shown in Figure 8 indicates that the application of nonlinear SSI analysis for the beyond design-level seismic analyses is of key importance to obtain reasonable ISRS estimates for the performing meaningful structure and equipment seismic margin and fragility analysis evaluation. Unfortunately, at this time, many "expert" practitioners overlook to address these important nonlinear structure dynamic behaviour aspects in a reasonable way for the fragility analyses, although efficient engineering tools to improve their crude assumptions are available. However, the nonlinear SSI analysis has to be done right. For example, assuming that the SSI effects are decoupled and, therefore, they are not influenced by the structural nonlinear behaviour with the argument that the foundation SSI motion does not change (not true!) that is a wrong assumption which surprisingly was repeatedly used in several papers published in "prestigious" engineering journals and conferences as a state-of-the-art nonlinear SSI approach to the structural fragility analysis.

REINFORCED CONCRETE CONTAINMENT PUSH-OVER CASE STUDY

This case study is of particular interest for showing the application of the nonlinear SSI approach to perform a pushover analysis for calibrating the BBC used for the local containment nonlinear shell elements given that there are available experimental test data. The pushover analysis was performed for the fixed-base model. It should be noted that this containment configuration shown in Figure 9 (left) is similar to the “Surrey” reinforced concrete (RC) containment structure tested until failure at the DOE Sandia National Labs (SNL) under an increasing lateral static load as described in NUREG/CR-6783. The containment structure modeled is a typical containment design used in the US. The containment's geometric configuration and FE modeling are described in Figure 9.

The containment structure is modeled using shell elements as shown in Figure 9 (right). In the horizontal direction the containment shell (CS) model was divided into 24 segments in the circumferential direction around the circle, to limit the angle between shell planes below 15 degree that is acceptable FE modeling practical rule for accurately modeling curved surface walls using the flat thin shell elements formulated based on the Kirchhoff plate element theory. The elements size of cylinder part are about 17ft and 11ft in the horizontal and the vertical direction, respectively. The wall thickness is 4.5ft. Each shell element was considered to be a nonlinear wall panel with degrading stiffness due to the in-plane shear deformation. To split of the containment structure model in separate local shell elements the UNIPL command can be applied in a single step. The CS panels are the shell elements. Since the CS structure is a curved shell, the BBC for these curved wall panels was done iteratively.

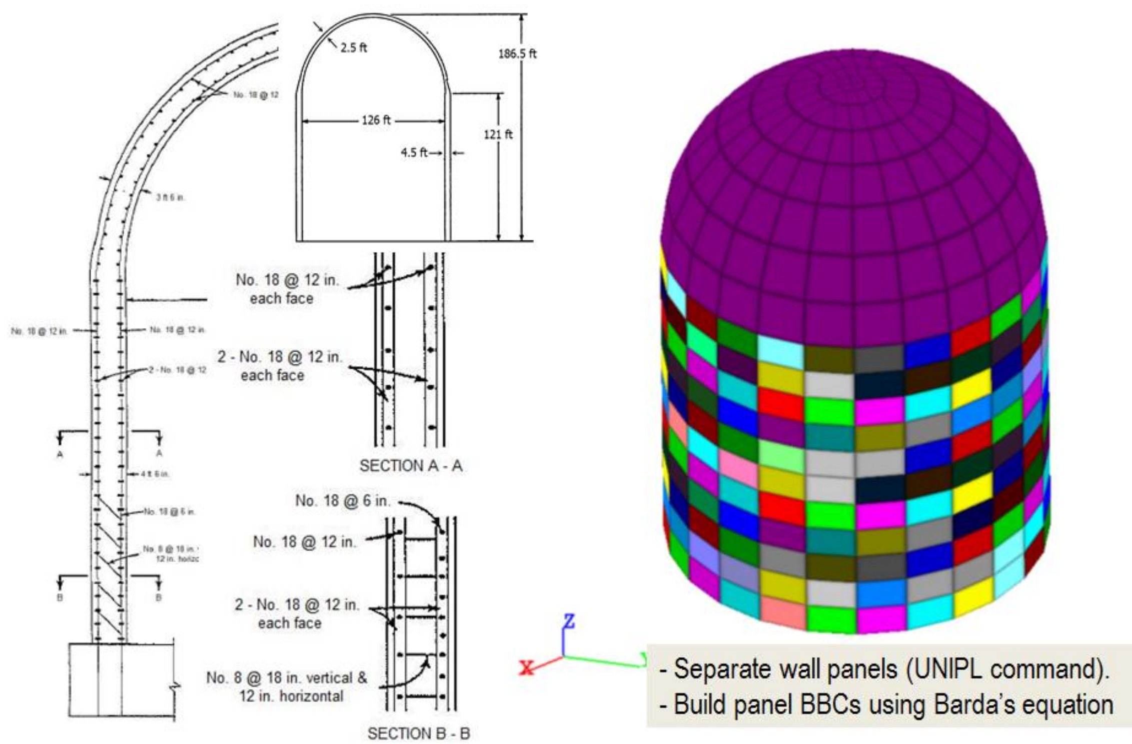


Figure 9 Containment Shell Geometry and Reinforcement (left), and FE Model for Pushover Analysis

To execute the pushover analysis a number of nonlinear static analysis are performed for increasing lateral force values. The nonlinear SSI analysis is done for 8 ground motion acceleration levels, 0.1g, 0.6g, 1.0g, 1.5g, 2.0g, 3.0g, 4.0g, and 5.0g. To simulate static loading in the ACS SASSI, the analyst needs to define a very slow-loading that produces no dynamic effects.

The nonlinear results can be improved by modifying iteratively the elementary panel BBC based on the overall pushover force-displacement relationship (global BBC of the CS structure system). Iteratively computed results are shown in Figure 10. The initial pushover run was done assuming the wall panel BBC were computed using the Barda 1977 shear failure equation implemented in ACS SASSI Option NON (2016). The initial or iteration 1 force–displacement relationship is plotted with a black line. To improve the matching between the computational results and the SNL test results for the pushover analysis, the elementary panel BBC shapes were adjusted by multiplying them by the ratio between the SNL test pushover curve and the CS structure pushover curve obtained from analysis. As shown in Figure 10 for only 3 iterations, the procedure based on modifying iteratively the elementary panel BBCs using the global CS pushover curve is fast convergent.

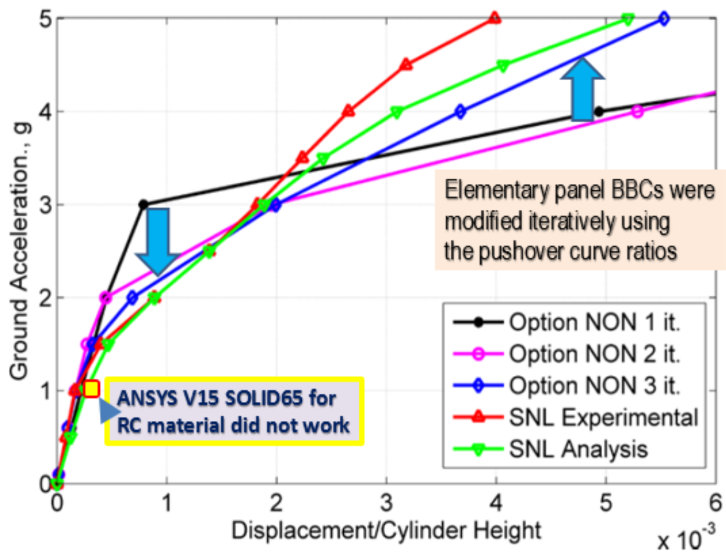


Figure 10 Comparison of ACS SASSI Option NON Pushover Results for Iteration Runs 1, 2 and 3 with the Sandia Labs Test and ANACAP Pushover Results in NUREG/CR-6783

CONCLUSIONS

The paper illustrates the application of the fast nonlinear SSI analysis hybrid approach that uses an efficient iterative equivalent-linearization procedure. Application of the nonlinear SSI approach to a low-rise concrete shearwall building is demonstrated in the light of the new ASCE 4-16 standard recommendations for the design-basis level (DBE) and the beyond design-basis level (BDBE). The paper also shows the application of the nonlinear approach to a pushover analysis of a concrete containment structure for calibrating the concrete force-displacement relationship based on experimental tests results.

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